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Professor A. J. S. Pippard, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Structural Paper No. 35

"The Design of a Reinforced-Concrete Factory at Brynmawr, South Wales"

by

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SYNOPSIS

The industry for which this factory was required demanded a structure somewhat out of the ordinary and so a deliberate attempt was made to satisfy industrial, architectural, and engineering desiderata by placing the design in the hands of a team representing these different aspects.

After mentioning previously published general descriptions and describing functional requirements briefly the Paper deals in detail with some selected points of engineering interest :—

(1) Ground survey and the evidence from which foundation requirements were deduced.
(2) The operation of filling with cement grout cavities left from old coal workings.
(3) Bored-pile foundations and the method of supporting lightly loaded columns on single untied piles.

(4) The location of expansion joints and a detailed description of the method used to obtain thermal isolation between the nine domes of the roof of the main production area.
(5) The principles adopted in the design of the rectangular shell domes with an explanation of why something more than a rough approximation was required.
(6) Design considerations for the bow-string girders forming the boundary supports of the domes. A description is given of the theoretical analysis adopted and, for the purpose of illustrating the practical application of the theory, much of the numerical work has been reproduced.

Introduction

The construction of the factory for Brynmawr Rubber Limited was begun in 1947 and completed in 1950. It was preceded by the boiler-house on an adjacent site. Much of the publicity given by the technical press to this factory has been based, apparently, on the material later brought out in booklet form by the Cement and Concrete Association.[1] The Authors have therefore assumed that those interested in a general description will avail themselves of this publication, and in the present Paper have concentrated on some points of engineering interest which have hitherto received little attention. An architectural assessment has appeared in *Architectural Review*.[2]

Although most of the information (but not opinion) contained in previous publications has been supplied by the architects and consulting engineers, this Paper is the first by any of those directly concerned and its introduction is therefore chiefly devoted to background material necessary for a critical appraisal of the structure.

Bringing this new industry to Brynmawr was but one of the practical steps initiated and carried through by the Earl of Verulam in the rehabilitation of the valleys of South Wales. The area was derelict in more senses than one, because the working-out of the coal that had originally brought people there had left most available sites liable to subsidence. Between the wars such sites were usually left strictly alone. At the present time, however, building operations in many parts of South Wales are extending to them increasingly. The Authors are of the opinion that all experience in dealing with the problems involved, whether successful or not, should be made known, and the Paper begins with an account—as factual as possible—of the steps taken in this instance of shallow workings.

Planning for the production, services, and administration of the factory was begun towards the end of the 1939–1945 War as part of the movement afoot at that time to prepare for the anticipated transition from war- to peace-time industry. It was conducted by a team comprising a group of architects, the Architects' Co-operative Partnership, and industrialists led by the managing director, Lord Verulam, of the company that was to operate the factory. No structural decisions were taken until the team

[1] The references are given on p. 373.

had been augmented by calling in the consulting engineers shortly after the end of the war, by which time a fairly clear picture of the functional requirements had emerged.

Briefly, the whole of the factory was to be air-conditioned ; the long rooms to the north and west were for more or less fixed and predetermined plant used in the production of sheets and other strips from crude rubber ; the south block was for cloakrooms, administration, ventilating plant, and work-shops ; the remainder of the factory area, called Main Production, had to have the maximum flexibility with no internal columns in the three production-belt bays running from south to north. The only major function then positionally undetermined was that of Printing and Spreading. Because of its higher fire risk this department was ultimately housed in the separate building to the south of the factory. On the industrial side there was experience in the rubber manufacturing industry, and the most important requirement affecting the structural form was that there must be an absolute minimum of dust-collecting surfaces, especially those liable to vibrate, such as ventilation trunking.

Naturally, this requirement considerably narrowed down structural possibilities, but sufficient of them were considered to make the engineers appreciate that, with a client with the vision to want the best possible working conditions to attract operatives and to contribute to high productivity, and with architects bent on realizing these aims, the process of selection of economical structural forms was more deliberate and quantitative than is usual in industrial building.

With the additional factor of using as little steel as possible, it soon became apparent that shell-concrete roofing over the longer spans was the best solution to these requirements. Cylindrical shells are today coming into the category of ordinary designs and are not mentioned in this Paper except for an interesting point in connexion with the roof of the main entrance hall at the south-east corner.

For the Main Production Area the requirements already mentioned and that of good natural lighting led the team to adopt the nine rectangular domes with walk-ways between them. With this arrangement there were only four groups of supports in the interior of the area and the ventilation trunking and other services were housed out of sight between the tie-beams of the bow-string girders forming the boundaries of the longer sides of the domes.

These dome structures were to the Authors a novel embodiment of shell concrete. Shells are fairly flexible structural forms and well able to take up temperature changes by deformation, but their boundary members usually are not. The method of dealing with expansion joints in the roofing of the Main Production Area is described. With regard to flexibility in this area for possible frequent changes of production lines, its flat slab floor over the storage basement below was provided with a grid of holes, each with a cast-iron cover, so that electric power, etc.,

could be brought up from the main distributors below at any desired positions. The columns for this flat slab floor, where not combined with the roof supports, were lightly loaded and their piled foundations are described. The basement floor was isolated from the foundations and rested on the ground.

Although this factory was completed only 3 years ago, the time when the Authors were engaged on its design seems very much in the past. Today they would make use of post-tensioning in the shell boundaries and if called upon to give an opinion of the factory as a whole, would question whether the narrow cylindrical shells were the best solution to the roof of the northern strip. They confess that this feature took the longest to decide and produced the greatest number of alternatives for consideration.

Ground Survey

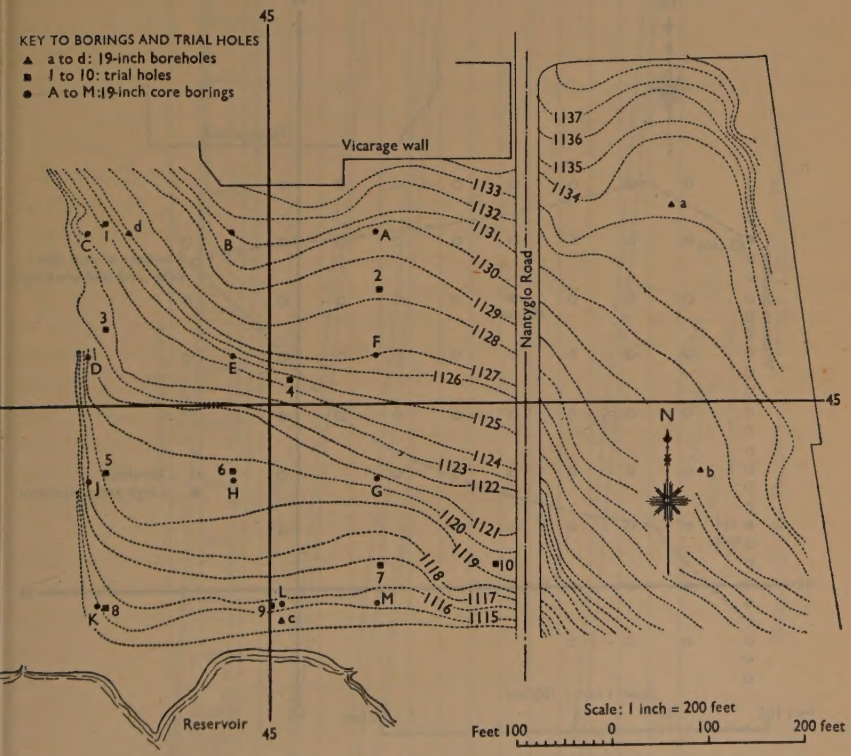
Brynmawr lies more than 1,000 feet above sea level at the head of the Ebbw Fach valley, which runs north from Newport in Monmouthshire. The base of the South Welsh coal measures breaks surface to the north of the town and the coal seams dip southwards to the coast at an inclination of about 5 degrees. Below the site of the factory at the southern edge of the town there are thus only three workable seams, but they are all near the surface and the ground is penetrated by a shallow labyrinth of mine galleries. Most of these have long since been worked-out or flooded and abandoned. Only one colliery is now left working in the district. Surface subsidence is common to the whole area and it was clear from the beginning of the project that something would have to be done to safeguard the structure of the factory from differential settlement (*Fig. 1*).

The first step was to find out the extent of the old workings beneath the factory site. Maps were obtained from H.M. Inspector of Mines which showed plans of workings in the Yard Vein and the Old Coal Vein. Depths were uncertain, but information from the Geological Survey and subsequent trial borings established the Old Coal workings as the deepest at about 130 feet below ground level, with the Yard Vein above this at 40 to 60 feet and the Meadow Vein even nearer to the surface (*Fig. 3*). Pillar-and-stall workings in the Old Coal Vein extended beneath the whole of the site and had been abandoned in 1878. The Yard Vein workings overlying this penetrated the site from two sides. On the east side the vein had been worked by long-wall mining (abandoned in 1890) and on the west side by the pillar-and-stall method. A shaft to the eastern long-wall galleries was found on the site. The Meadow Vein had apparently not been worked in this area and therefore could be ignored. Subsidence risks depend on the method of mining, the depth of the seam, and the length of time which has elapsed since the extraction of the coal. In longwall mining most of the coal is removed and the galleries extended by supporting the roof with

props. When working is completed the miners retreat and salvage the majority of the props as they go. As the remaining supports decay the roof collapses and, if the workings are no deeper than 600 feet, the final subsidence to the extent of 60 per cent of the seam thickness will have taken place after about 2 years.

The consequence of the pillar-and-stall system of getting coal are much less certain. Here pillars of coal are left to support the roof of the galleries.

Fig. 1


KEY TO BORINGS AND TRIAL HOLES
▲ a to d: 19-inch boreholes
■ 1 to 10: trial holes
● A to M: 19-inch core borings
Vicarage wall
Nantyglo Road
Reservoir
45
N
Scale: 1 inch = 200 feet
Feet 100 0 100 200 feet

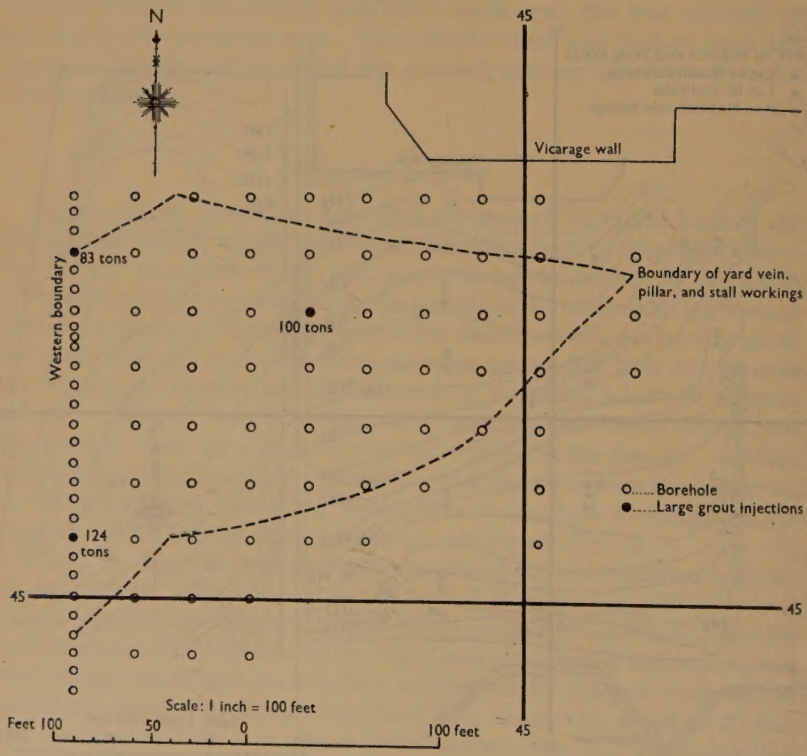
Plan of Site and Ground Survey

If the coal seam is based on a softer rock the pillars may gradually sink into the floor of the gallery; on the other hand, the pillars may be eroded by moisture or oxidization, and in either case the amount of subsidence will depend on the proportion of coal which has been taken away. Obviously, subsidence may be indefinitely prolonged.

It follows from this that the area of the site with the greatest subsidence risk was that over the Yard Vein pillar-and-stall workings of 1879, which

encroached on the site from the west (*Fig. 2*). The treatment of this area is described in a later section of the Paper. The other Yard Vein workings under the east side of the site were long-wall galleries abandoned in 1890, and it was therefore assumed that the final settlement had already taken place and the ground was stable. This conjecture was borne out by the fact that Brynmawr Railway Station, which stands upon another part

Fig. 2



GROUND TREATMENT—GROUTING

of these same workings, is known to have sunk several feet in 1892. Finally, it was assumed that there was little to fear from the Old Coal workings which, although they had been worked on the pillar-and-stall system, were felt to be deep enough and sufficiently well distributed under the whole area of the projected building to cause only slight subsidence.

Four 6-inch trial borings (shown by the black triangles lettered a, b, c, and d on *Fig. 1*) were driven by a firm of specialist contractors to a depth of 100 feet to check the levels of the local coal seams and to discover the

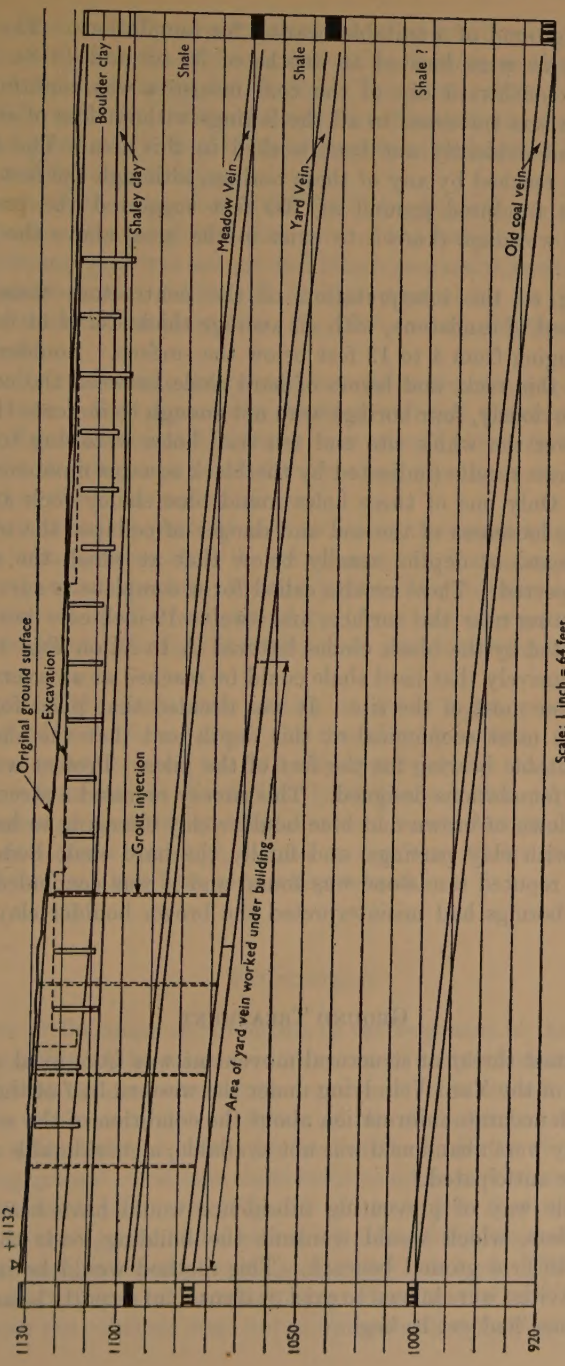


Fig. 3

North–South Section on Typical Line of Piles

depth below ground of a suitable bearing for foundations. The Meadow and Yard Veins were located at depths of 30–58 and 48–84 feet and the 5-degree southward dip of the coal measures was confirmed. The Meadow Vein was traversed in all the borings without loss of circulating water and had evidently not been worked in this area. The Old Coal Vein was not reached by any of these borings, although the fact that one borehole met fractured ground at 100 feet suggested the presence of old ironstone workings (known to exist in the area) above the Old Coal level.

According to the interpretation of the contractors these borings indicated a bed of sandstone, with an average thickness of 14 feet, found at depths ranging from 4 to 12 feet below the surface. Boulder clay was found above this rock, and layers of hard shale between the coal seams below it. Obviously, four borings were not enough to indicate the subsoil conditions over the whole site and ten trial holes were dug to test the validity of these results (indicated by the black squares numbered 1 to 10 on *Fig. 1*). Only one of these holes found blue shaley rock at 15 feet. Owing to the looseness of the soil and danger of collapse the others had to be abandoned at depths usually below that at which the sandstone had been expected. These results called for a complete re-survey of the ground structure near the surface, and twelve 19-inch-core borings were made (indicated by the black circles lettered A to M on *Fig. 1*). These showed conclusively that hard shale could be reached at an average depth of 14 feet over most of the site. It was decided that piled foundations would be the most economical at this depth and that the shale would provide a suitable bearing for the feet of the piles. Profiles were drawn and suitable foundations designed. This survey revealed a structure from the surface down of brown and blue boulder clay changing to hard shaley clay, shale with clay partings, and finally the hard shale bedrock. No trace of the reputed sandstone was found and it was concluded that the earlier trial borings had misinterpreted the brown boulder clay as sandstone.

Ground Treatment

The greatest threat of structural movement was from local subsidence in that part of the Yard Vein lying under the western half of the building, and although accurate information about the condition of the workings at the time they were abandoned was not available, a considerable number of cavities were anticipated.

A possible way of preventing subsidence would have been to grout concrete pillars, which would transmit the building loads through the cavities on to firm ground beneath. This method would be indicated if very large cavities were known to exist or if coal had recently been extracted and subsidence had yet to begin.

In the case of the Brynmawr site the seam was shallow and the workings more than 50 years old. It was reasonable to suppose that the cavities would be small and evenly distributed. Therefore, rather than inject stiff grout locally under the points of concentrated load, it was decided to inject a colloidal grout which would flow underground and, it was hoped, fill the existing cavities completely.

The area of the suspected Yard Vein lying beneath the site was 7,000 square yards—about half the plan area of the building. Over this area a 30-foot-square grid was set out and 2-inch percussion borings were made at the intersections. Additional borings were made at 10-foot centres down the western boundary (*Fig. 2*). The average depth of the borings was 63 feet.

Normally, circulating water was lost above the suspected level of the workings which had apparently subsided and left cavities in the overburden. For this reason the bores were carried down further until the level of the workings was reached. Grouting was begun at this level and continued as the tube was withdrawn.

A grout mix of $1\frac{1}{2}$ parts of sand to 1 part of cement was injected into each bore at a pressure of 50 lb. per square inch, increasing to 250 lb. per square inch when the hole became tight. The western boundary was injected first so that a wall would be formed to prevent the escape of subsequent grout injections into the main workings lying to the west of the site.

A total of 119 bores were made; in 43 of these, cavities were encountered, but only three were of any considerable extent. The estimated quantity of grout injected was 779 cubic yards, which was 15 per cent of the total volume of the Yard Vein lying beneath this part of the site. In the course of grouting operations several bores were made outside the assumed area of the workings, but no cavities were found and it appeared that the boundary had been accurately defined.

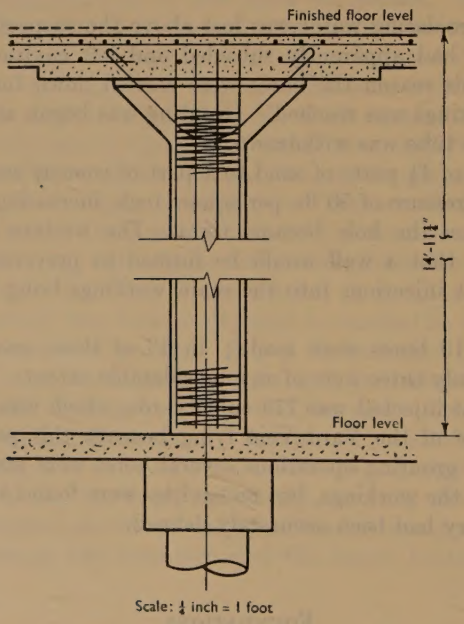
Foundations

In view of the space requirements in the basement of the factory and the floor loading of the main production area, a square structural grid of 19-foot-$1\frac{1}{2}$-inch spacing was selected as the most economic span for the mushroom column-and-slab structure of the main production-area floor. This gave a load of 62 tons at the base of the majority of columns. In the existing ground conditions these loads could be carried on single bored piles of 19 inches diameter (*Fig. 4*). It was clear that a considerable economic advantage could be gained by using fully stressed single piles rather than understressed pile groups to carry this load. This implied an unusually high degree of accuracy in positioning the piles if they, and consequently the columns, were not to be seriously out of line on one hand

or submitted to excessive flexural stress due to eccentric loading on the other.

The permissible bending stress in the head of the pile and the foot of the column limited the eccentricity to 1½ inch. Consultation with the piling contractors revealed, however, that with the driving equipment at their disposal they could not guarantee a smaller maximum displacement than 2 inches. It was agreed that, when any displacement up to 1½ inch of the column base on the pile occurred, the pile cap was to be hacked back and the starter bars cranked into the correct position in situ. If a

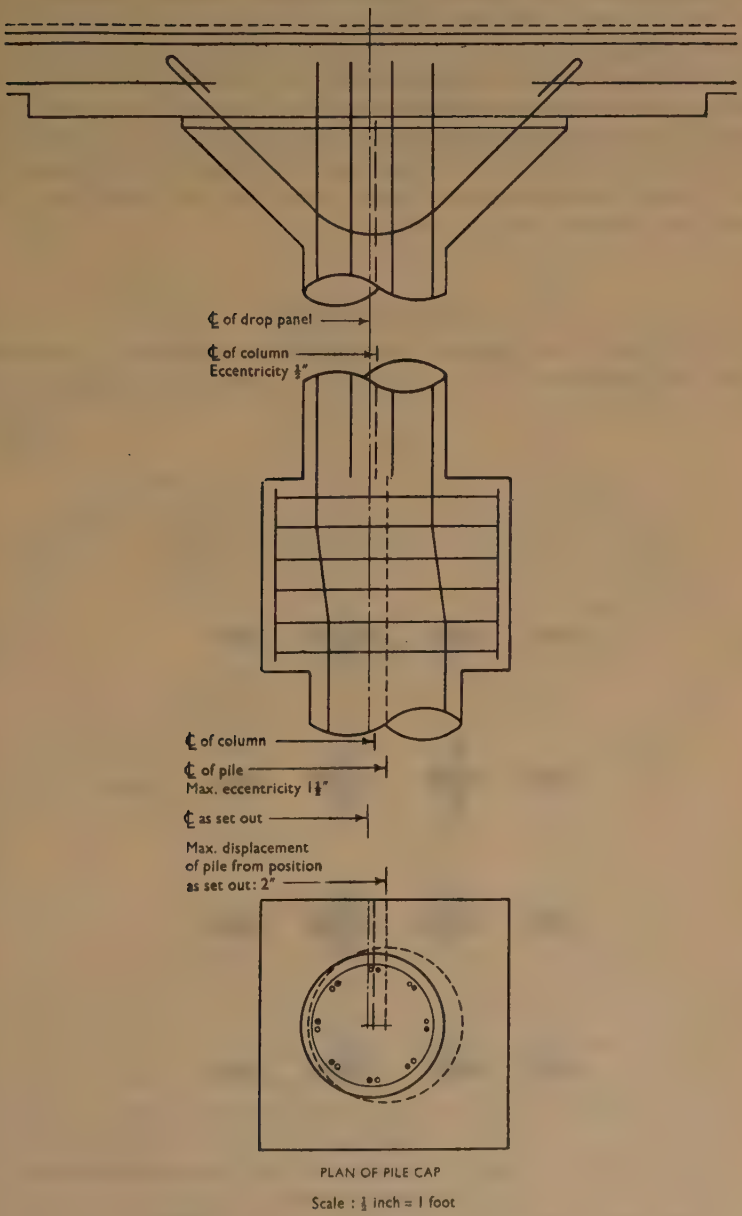
Fig. 4



displacement of between 1½ inch and 2 inches occurred, the column could be moved up to ½ inch off-centre and the column head correspondingly displaced on the floor-slab drop panel which would remain accurately in position (*Fig. 5*). The contractors agreed that if the pile displacement was more than 2 inches they would drive an additional pile free of charge.

The technique proved to be successful. Out of 270 single piles driven to an average depth of 13 feet 6 inches, 251 (93 per cent) had a displacement of less than 1½ inch ; 17 (6 per cent) had a displacement of between 1½ inch and 2 inches and the corresponding columns had to be moved. Only 2 (approximately 1 per cent) of the piles were more than 2 inches out of position, and in these cases the contractor drove additional piles.

Fig. 5



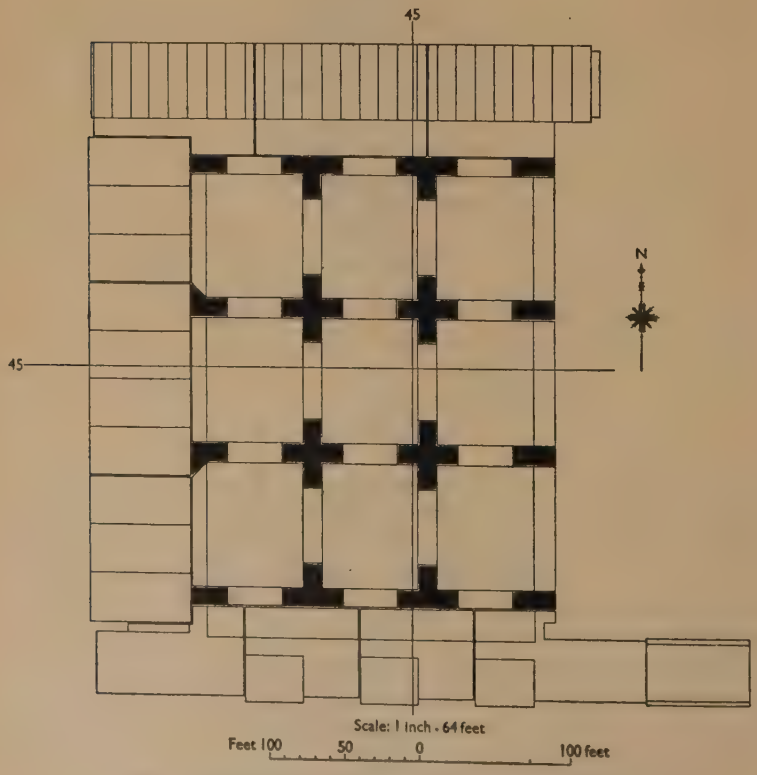
Arrangement with Pile 2 inches out of Correct Position

Bored piles were used so that the nature of the ground at the foot of the borings could be inspected and an adequate bearing ensured before the pile was cast. After the shale had been reached the boring was extended a further 12 inches. *Fig. 3* shows a section on a typical line of piles and compares the levels reached by the pile bases with one of the profiles constructed from the final series of trial borings. Apart from the inevitable inaccuracies of the profiles, the larger discrepancies where the piles stop short of the shale bed are probably caused by the presence of grout injected into subsidence fissures above the coal seam level and forming an artificial footing to the piles.

Expansion Joints

In this part of the Paper the building will be considered in its fundamental structural elements : a multi-domed core with vaulted galleries to

Fig. 7



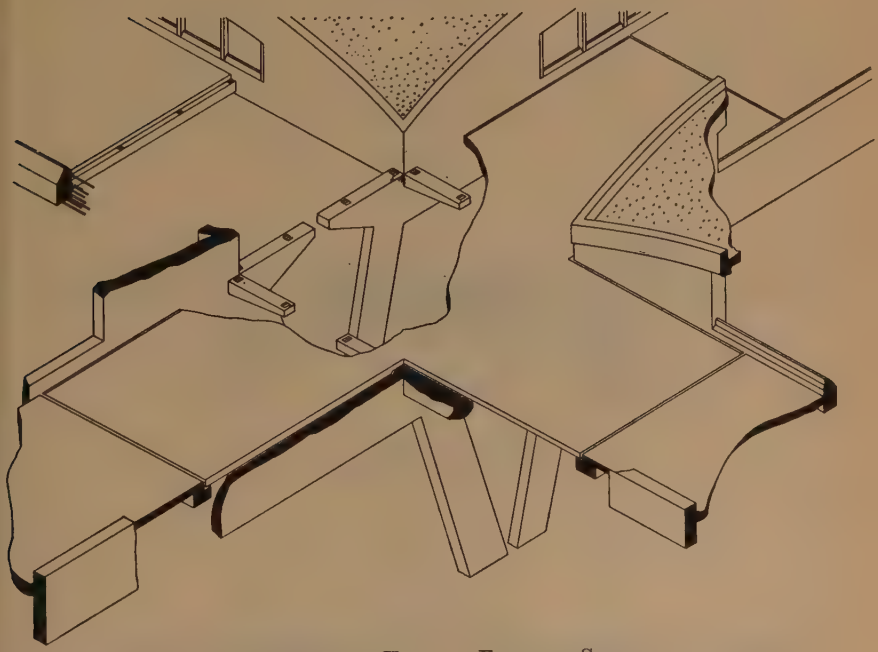
Articulation—General Arrangement

the north and west and a cellular complex to the south whose most easterly compartment is roofed with a large single vault (*Fig. 6*, facing p. 368).

It is obvious that the resolution of problems of thermal and other movement in such dissimilar elements will result in expansion joints of some complexity, particularly since the designer had constantly to bear in mind the possibility of differential settlement over the worked-out Old Coal Vein, against which no grouting precautions had been taken (*Fig. 7*).

The main roof is structurally continuous, the nine domes being mono-

Fig. 8



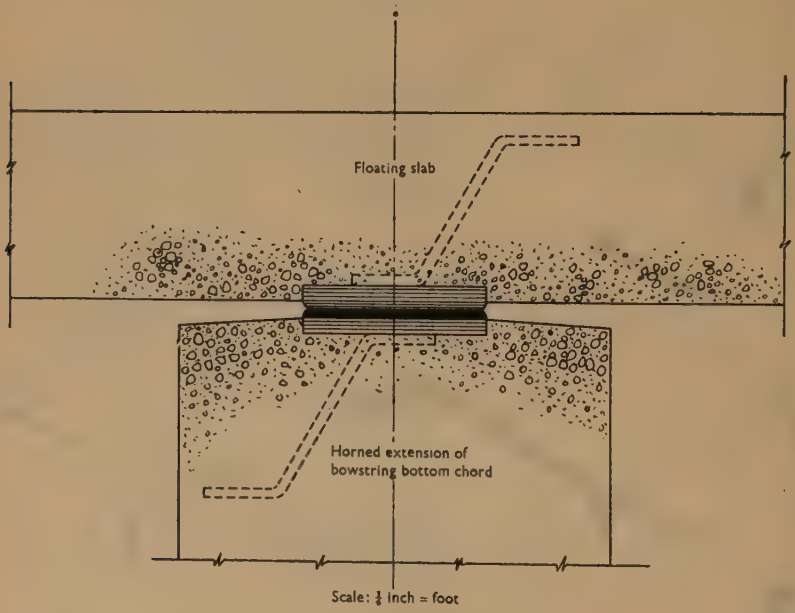
Axonometric View of Floating Slab

lithic with their dividing walkways. All serious thermal stresses are eliminated by the domical configuration of the surface; the relatively small thermal movements of the domes themselves is taken up by flexure of the thin hanger members of the edge bow-strings. Movement in the walkways between the domes is taken care of by "floating" cruciform slabs at the walkway intersections. These slabs not only break up the length of the walkways but also allow the dome corners to expand and contract at the point where the bow-strings are too shallow and too stiff to permit the lateral flexure that is possible along the centre part of the span (*Fig. 8*).

This " floating " slab is supported on horned extensions of the bottom chord of the bow-strings and on rebates in the ends of the " fixed " walkways. On each horned projection and on each rebate are sets of phosphor-bronze pads, lined on the upper surface with graphitized bearing metal. These pads are paired with unlined phosphor-bronze pads, polished on the lower surface and cramped into the soffits of the " floating " slabs (*Fig. 9*).

Articulation of the subsidiary vaulted galleries and the multi-domed core is maintained through the various levels with either lightly loaded

Fig. 9



Sliding Bearing

Upper pad : 4 inches by 3 inches by ½ inch unlined polished phosphor-bronze
Lower pad : 4 inches by 3 inches by ⅜ inch phosphor-bronze with ⅛ inch graphitized bearing-metal lining

horizontal " paper " joints or vertical gaps packed with jointing material. These joints are not all in the same vertical plane and the gaps shown between the parts in *Fig. 7* are merely diagrammatic.

The south cellular complex is separated from the core and divided into four parts by similar combinations of conventional expansion joints.

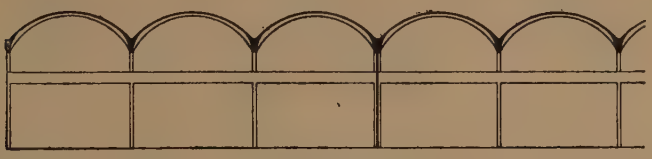
The western vaulted section has two expansion gaps between vaults 3, 4, 7, and 8, the supporting columns at these points being split (*Fig. 10*).

In the northern vaulted section the problem of thermal and other movements was more complicated. The roof with its light gable frames is capable of taking expansion in the shell, but the heavy substructure

(crane-beams, supports, etc.) must be discontinuous. Expansion gaps were therefore provided in the crane-beams in the centre of the 9th and 18th spans (*Fig. 11*).

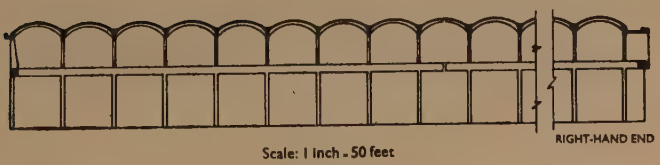
Increased flexibility in the vault over these expansion joints was achieved by splitting the gable frame at the crown and hinging the shell by stop-ending one-half of the shell in short alternating oblique sections and casting the other half up to this keyed edge (*Fig. 12*). The shell is

Fig. 10



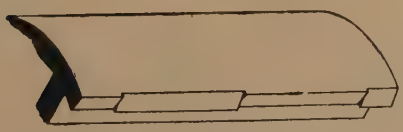
HALF SECTION—WEST VAULTS

Fig. 11



PART SECTION—NORTH VAULTS

Fig. 12



HINGE AT CROWN OF VAULT

in circumferential compression at the crown and no opening and closing of the joint is visualized—the shell is simply given freedom to change direction. The shortness of the vault chord makes possible the cantilever of the split gable frame and the keying of the hinge prevents relative vertical displacement of the two halves of the shell.

The winged profile of the eastern vault presented a special problem because calculations showed that downward and inward deflexion would take place along the edges of the shell, the crown remaining level (*Fig. 13*).

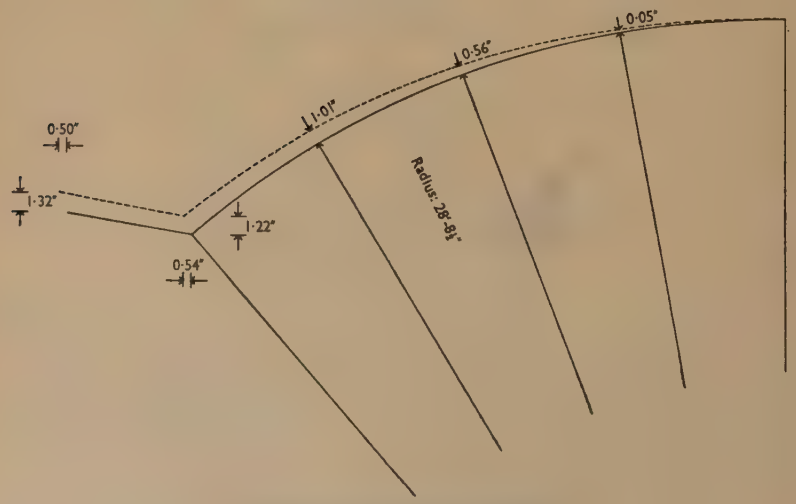
Fig. 13

East Vault—Profile

To maintain a level soffit the shuttering was constructed with an upward camber and struck in a sequence corresponding to the varying deflexions along the length of the shell (*Fig. 14*).

The changes in level involved were large enough to be checked by the admittedly rough method of taking levels. These changes not only agreed with the calculated results but when re-taken at monthly intervals indicated no appreciable change ; this revealed that the stresses were in the elastic range and low enough for creep to be a negligible quantity.

Fig. 14



Camber—Entrance-Hall Shell

Design of Rectangular Domes

It has already been explained in an earlier section that through the method of interrupting structural continuity, each of the nine domes in the main production area is virtually an independent structure. In view of the interest in reinforced-concrete shells, the Authors take this opportunity to record the method they used to design the complete dome structure from the main production floor level upwards, with as much numerical work as space will allow. The problem was divided into two parts: the rectangular shell-concrete dome itself, and the boundary members comprising bow-string girders with their supporting columns. These parts act together and form one structure.

In considering an actual example, the importance of adopting design considerations for the shell which make it possible to obtain a reasonable design for the boundary members will receive more emphasis than in much of the theoretical literature on shells, which are often treated as though they were in themselves complete structural entities. In fact, with this extremely stable double-curved form, in which the shell stresses are low and mainly compressive, there is no doubt that much rougher and more approximate considerations could have been adopted to obtain substantially the same amount of reinforcement required across the tensile zones in the corners.[1] Large spans in reinforced concrete usually require some comparatively heavy reinforcement somewhere, and in this case it occurs in the tie members of the bow-string girders. These, therefore, rank on the grounds of safety and economy as the major problem. Frequently the real reason for developing shell theory and using computation methods such as relaxation for shell stresses is to gain a fairly accurate idea of the manner of distribution of load from the shell to the boundary members.

Rectangular Shell-Concrete Domes

The design assumptions will be stated as briefly as possible without unnecessarily repeating explanations to be found elsewhere. With the exception of regions near the boundaries, the load on an element of shell is supported by direct stresses and the major part of the design assumed that the shell possessed no strength in bending. As a matter of convenience the analysis of these direct stresses was transformed to a horizontal plane, sometimes called the affine shell (see *Fig. 15*), in which case the stress resultants may be expressed in terms of Airy's stress function:

$$N_{xx} = \frac{\partial^2 F}{\partial y^2}$$

$$N_{xy} = N_{yx} = -\frac{\partial^2 F}{\partial x \partial y}$$

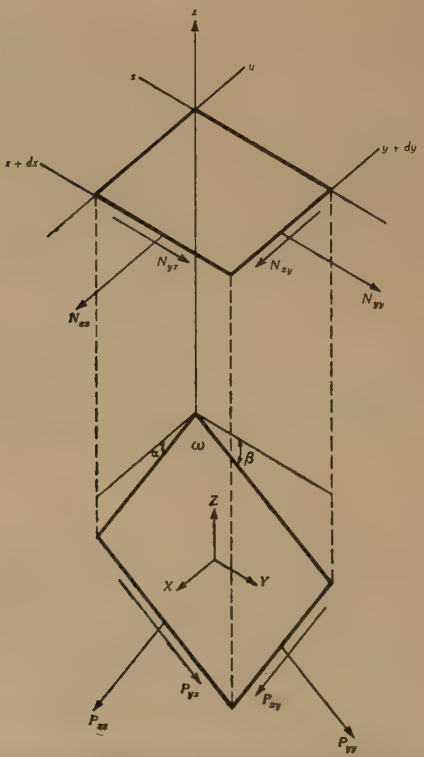
$$N_{yy} = \frac{\partial^2 F}{\partial x^2}$$

In this form, the horizontal equations of equilibrium are satisfied, leaving the vertical equation of equilibrium to be solved :

$$\frac{\partial^2 F}{\partial x^2}\frac{\partial^2 z}{\partial y^2} - 2\frac{\partial^2 F}{\partial x \partial y}\frac{\partial^2 z}{\partial x \partial y} + \frac{\partial^2 F}{\partial y^2}\frac{\partial^2 z}{\partial x^2} + Z\sqrt{g} = 0.$$

where z denotes ordinate, Z denotes vertical load per unit area, and $\sqrt{g}$ denotes ratio of area of element in space to area on plan.

Fig. 15



Stresses on a Shell Element projected on to a Horizontal Plane

The horizontal loads from wind, etc., have been omitted because it has been found that the only significant load in a double curved shell of this kind is the vertical one. The derivation of this third equation of equilibrium has been given by one of the Authors as an example from the

general case in a recent publication [3] and also by direct considerations for this cartesian case by Johansen [4] and others. It should be clearly understood that this extensional (or membrane) theory merely satisfies equilibrium on the assumption of no flexural rigidities and makes no other assumption regarding the properties of the material.

The use of the stress function is sometimes criticized because its second derivatives define the stresses, which are therefore much less accurate than the values of the function itself when finite-difference methods are employed. It has already been mentioned, however, that the aim is not to find the stresses in the shell with any high degree of accuracy. They are subordinate to the boundary structures for which the condition of no appreciable lateral load is satisfied by $F = 0$ on the boundary, and the total horizontal component of tension in them is given by the slope of the stress function perpendicular to the boundary. From this the load distribution from the shell to the boundary members is determined, to a higher degree of accuracy than stresses in the shell.

The shape adopted for the domes was a surface of translation generated by moving one vertical circle along another. This is shown in *Fig. 16*, which also shows the relaxation net in a horizontal plane in one quadrant —sufficient to solve a case with two axes of symmetry. The Authors do not propose attempting to explain how the method of relaxation is used in solving the type of differential equation under discussion, because this has been done by much more competent authorities. [5, 6] It should be mentioned, however, that, as with most of the more refined techniques in computation, it usually pays to undimensionalize the quantities involved. In the case of this circular translation surface :

$$\frac{\partial^2 z}{\partial y^2} = -\frac{1}{r\cos^3\beta}; \quad \frac{\partial^2 z}{\partial x \partial y} = 0; \quad \frac{\partial^2 z}{\partial x^2} = -\frac{1}{R\cos^3\alpha}$$

For simplicity, write $Zr = -1{,}000$,

which gives, $$-\frac{\partial^2 F}{\partial x^2} - \frac{r}{R}\cdot\frac{\cos^3\beta}{\cos^3\alpha}\cdot\frac{\partial^2 F}{\partial y^2} - 1{,}000\,\frac{\cos^2\beta}{\cos\alpha}\sin\omega = 0.$$

The governing equation in this form was actually used though later experience has shown that there is a slightly more convenient form for this case. Undimensionalizing is carried out by the following substitutions.

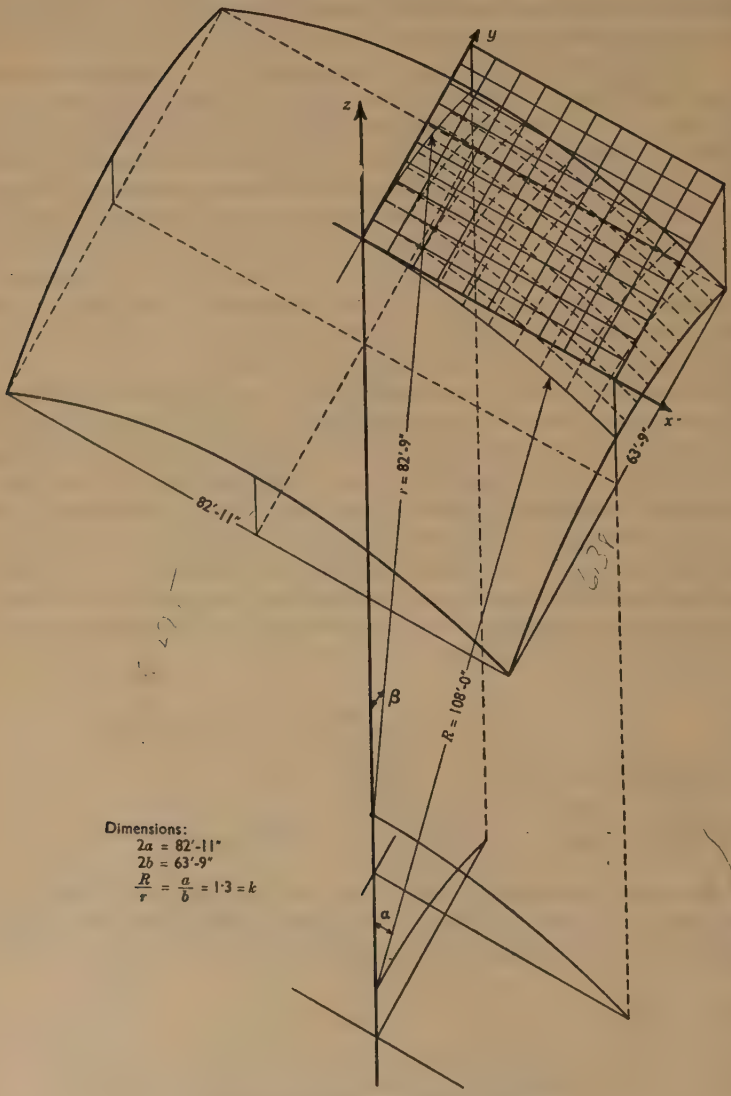
Let $x = \frac{a}{10}X$; $y = \frac{b}{10}Y$; $F = \frac{a^2}{100}f$,

then $$\frac{\partial^2 F}{\partial x^2} = \frac{\partial^2 f}{\partial X^2}; \quad \frac{\partial^2 F}{\partial x \partial y} = \frac{a}{b}\cdot\frac{\partial^2 f}{\partial X \partial Y}; \quad \frac{\partial^2 F}{\partial y^2} = \frac{a^2}{b^2}\cdot\frac{\partial^2 f}{\partial Y^2},$$

and the vertical equation of equilibrium becomes :

$$-\frac{\partial^2 f}{\partial X^2} - \frac{r}{R}\cdot\frac{a^2}{b^2}\cdot\frac{\cos^3\beta}{\cos^3\alpha}\cdot\frac{\partial^2 f}{\partial Y^2} - 1{,}000\,\frac{\cos^2\beta}{\cos\alpha}\sin\omega = 0.$$

Fig. 16

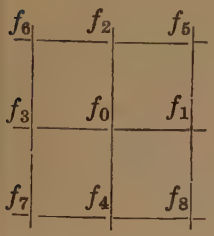


Shape and Dimensions of Dome

In this case $\frac{R}{r} = \frac{a}{b} = k$ and, writing $\frac{\cos\beta}{\cos\alpha} = \nu$,

$$\frac{\partial^2 f}{\partial X^2} + k\nu^3 \frac{\partial^2 f}{\partial Y^2} + 1{,}000\,\nu \cos\beta \sin\omega = 0.$$

This gives the finite difference equation and conversion factors for stresses.
Finite Difference Equation.



$$R \equiv (f_1 + f_3 - 2f_0) + k\nu^3(f_2 + f_4 - 2f_0) + 1{,}000\,\nu \cos\beta \sin\omega$$

where
$$N_{yy} = \frac{\partial^2 F}{\partial x^2} = (f_1 + f_3 - 2f_0)$$
$$-N_{xy} = \frac{\partial^2 F}{\partial x \partial y} = \frac{a}{4b}(f_5 + f_7 - f_6 - f_8)$$
$$N_{xx} = \frac{\partial^2 F}{\partial y^2} = \frac{a^2}{b^2}(f_2 + f_4 - 2f_0)$$

Fig. 17, Plate 1, shows the result obtained from the relaxation giving the undimensionalized values of the stress function at each mesh point together with the relaxation constants. As a matter of interest, the directions and magnitudes of the principal stress resultants were obtained as shown in Fig. 18, Plate 1, from which the stress trajectories in Fig. 19, Plate 1, were sketched. Fig. 20, Plate 2, shows the shell reinforcement obtained by this process.

This calculation ignored the circular openings in the shell for natural and artificial light. These openings were stiffened by kerbs which were judged to be at least adequate not to cause weakness. Because of these openings and a certain amount of steel for stiffening purposes the actual working drawing of the shell reinforcement was more complicated than shown in Fig. 20, Plate 2.

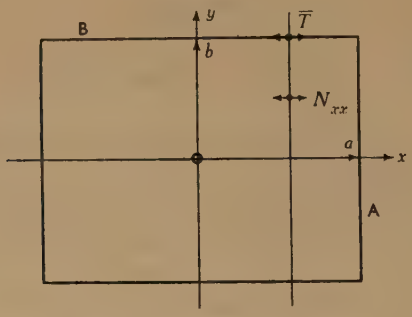
The Boundary Structures

Each edge of the dome is supported by a bow-string girder. A girder on a longer side is selected to illustrate the manner of loading and method of design. It is denoted by B in the plan shown in *Fig. 21.* From the first boundary condition of no lateral forces there must be a zero sum for the horizontal components of all stresses at right angles to any transverse vertical plane through the complete dome structure from its foundations upwards.

When $\bar{T}$ denotes the total tension in the boundary, this summation from the x axis of symmetry is given by :

$$\bar{T} + \int_0^B N_{xx} dy = 0.$$

Fig. 21



PLAN OF RECTANGULAR DOME

When the stress function is substituted it is seen that the total boundary tension equals the slope of the stress function :

$$\bar{T} = -\int_0^B \frac{\partial^2 F}{\partial y^2}\, dy = -\left[\frac{\partial F}{\partial y}\right]_B$$

In practice it is preferable to carry out the summation of the first expression by Simpson's rule rather than to use a finite difference formula for the second, because the former ensures that equilibrium is conserved —the primary consideration in the type of structural calculations which follow :

In accordance with the extensional solution, the load from the shell is transmitted to the boundary member by tangential shear forces along the edge. A difficulty arises in using this shear force directly. The assumed boundary condition of no outward force cannot be physically satisfied near the corners, and it saves going into approximate considerations of what actually happens there—as would be necessary in applying a finite difference summation formula to these shear loads—by basing the loading conditions of the boundary member on its total tension. The latter is at its maximum at the centre and diminishes to zero at the ends.

When $\bar{S}$ denotes the total vertical component of shear, owing to shell loads, in the boundary structure :

$$\begin{aligned}\bar{S} &= -\int_0^x N_{xy} \tan\alpha\, dx \\ &= -\int_0^x \frac{\partial^2 F}{\partial x \partial y} \cdot \frac{\partial z}{\partial x}\, dx \\ &= +\bar{T}\frac{\partial z}{\partial x} - \int_0^x \bar{T}\frac{\partial^2 z}{\partial x^2}\, dx\end{aligned}$$

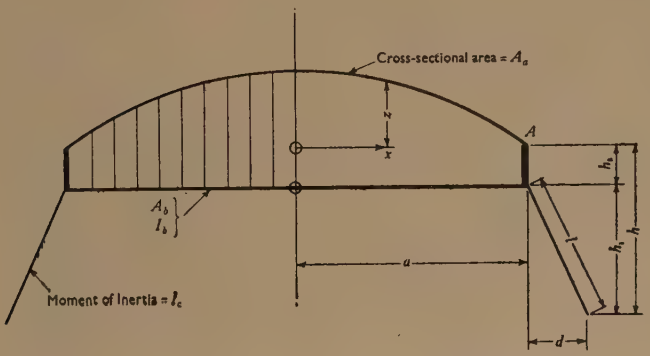
In the last form of this expression the vertical shear is obtainable from the already determined values of $\bar{T}$ and geometrical properties of the shape

in a form suitable for summation by Simpson's rule. The sum of the vertical shear at the ends of all the boundaries $\left(-\int_0^A T\frac{\partial^2 z}{\partial x^2}\,dx \text{ and similar}\right)$ must equal the total load on the shell. This check appears in the summations in Table 4, which compares with the load of the shell obtained as follows :—

Arc length, $R\phi$ = 42·60 ft
,, ,, $r\phi$ = 32·64 ft
Area of quarter shell = 1,390 ft² disregarding the small reduction due to the obliquities, ω.
Weight of quarter shell = 90 × 1,390 = 125,100 lb.

The boundary structure, shown diagrammatically in *Fig. 22* is statically indeterminate and was designed by the linear theory of structural analysis.

Fig. 22



Dimensions :

a = 41·458 feet
l = 15·0 feet
d = 3·75 feet
h = 16·0 feet
h_1 = 14·5 feet
h_2 = 1·5 feet

Cross-sections :
Arch : 1 foot 3 inches deep by 9 inches wide : A_a = 0·9375 sq. feet.
Beam : 3 feet 3 inches deep by 9 inches wide : A_b = 2·4375 sq. feet.
I_b = 2·1455 feet⁴.
Column : 2 feet 6 inches by 9 inches : I_c = 0·9765 feet⁴.

Loads :
Shell : 90 lb. per square foot, including superload.
Arch : w_a = 200 lb. per foot.
Tie : w_b = 1,000 lb. per foot dead load, including walkway plus 300 lb. per foot live load (Case II).

DIMENSIONS AND PROPERTIES OF SECTIONS (LONG SIDE)

The Authors were aware that such methods are nothing more than a ready way of finding a reasonable equilibrium solution that works in practice, and for this reason made further assumptions about this particular case with the object of avoiding lengthy calculations out of proportion to the premises of the design :—

(1) Because the tie member of the bow-string girder was much stiffer than the arch member, the latter was assumed to have no strength in bending, thereby reducing the structure to what is known as a flexible arch.

(2) Changes in length of the vertical hanger members caused by their stresses were assumed to be negligible.

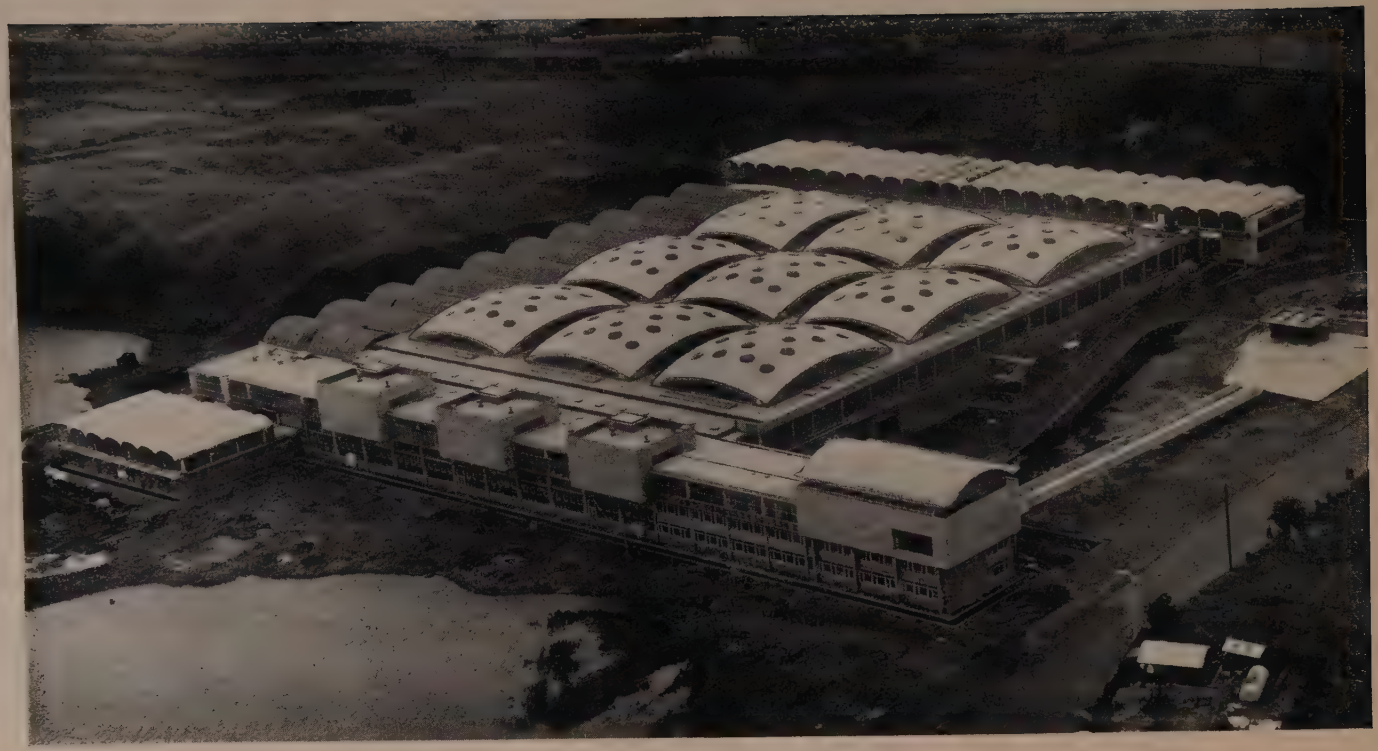
By these means a simple design with three degrees of statical indeterminacy in this symmetrical case was derived. Besides the tangential loads from the shell there were vertical loads from the self-weight of the boundary structure and the walkway slabs it supported.

At this point an explanation is required. In using an extensional solution for a shell dome and an elastic solution for the boundary structure it cannot be expected that the calculated stresses in the boundary arch will be the same as those along the adjacent shell edge. In reality the stresses along each side of this line of contiguity must everywhere be equal and therefore other types of shell stresses (bending and normal shear) must come into play in the edge regions. This action is called an edge disturbance. If the edge disturbance were not allowed for when the weight of the boundary is small, the arch member would appear to be in tension adjacent to a shell edge in compression. There are approximate methods of calculating these other types of shell stresses to remove this discrepancy.

In the case of domes, the disturbances do not spread far from the edge. In the case of cylindrical shells they penetrate much further and for this reason an elastic design for both the shell and the generating edge members is now usually adopted. Another reason is that rapid ways have been developed for applying this more rigorous design method to cylindrical shells. The difficulties about it for other kinds of shell structures are of a practical nature. The computation would be disproportionately long by the means at the present disposal of engineers.

In the case of the Brynmawr domes, the vertical loads supported by the bow-string girders were of the same order as the shell loads. The result was that, under the combined loads, the arch members were everywhere in compressive stress of the same order as the extensional shell edge stress. Edge disturbances were therefore not serious. The shell was slightly thickened at the edges and light top and bottom reinforcement for bending was provided in the edge regions. But to allow for the possibility of some of the shell load being transmitted vertically,

Fig. 6



Aerial View of Completed Factory

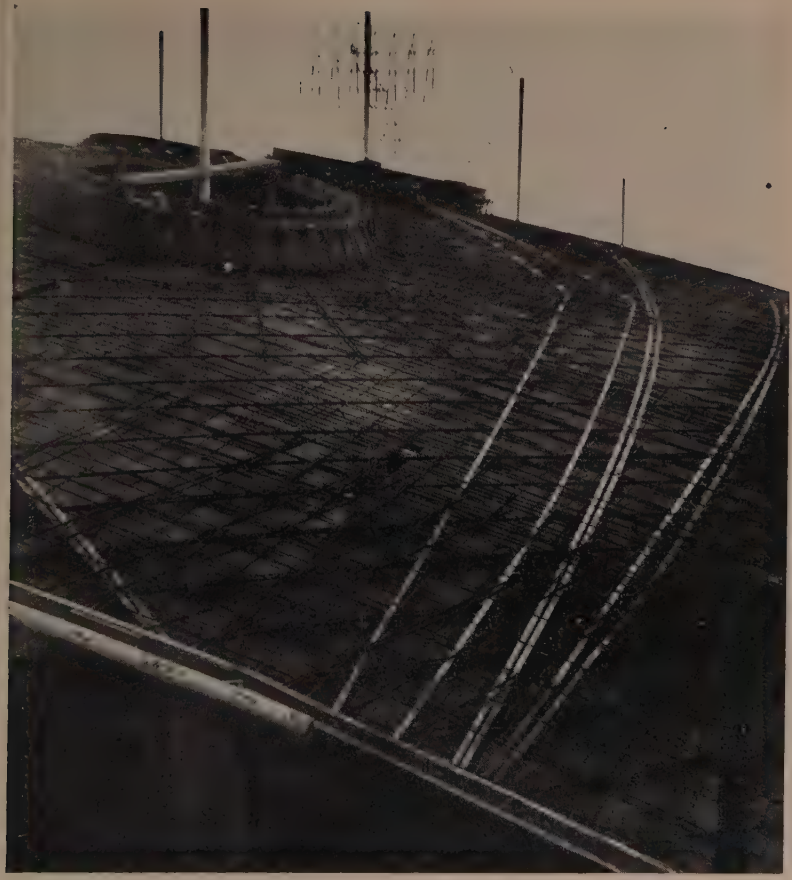
Fig. 26

BASEMENT OF MAIN PRODUCTION AREA

Fig. 27

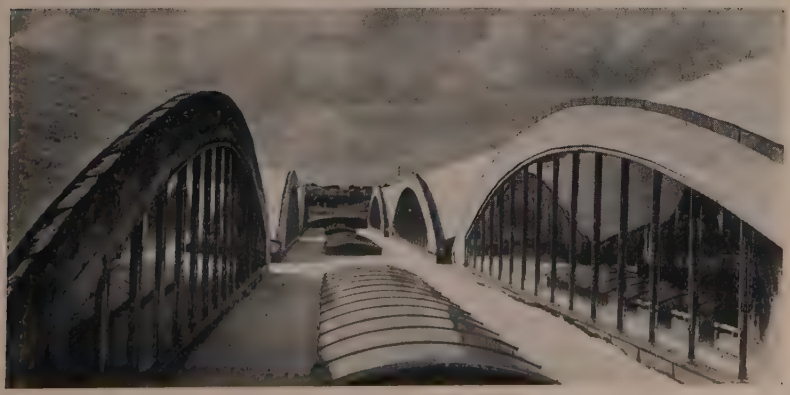
INTERIOR OF MAIN PRODUCTION AREA

Fig. 28



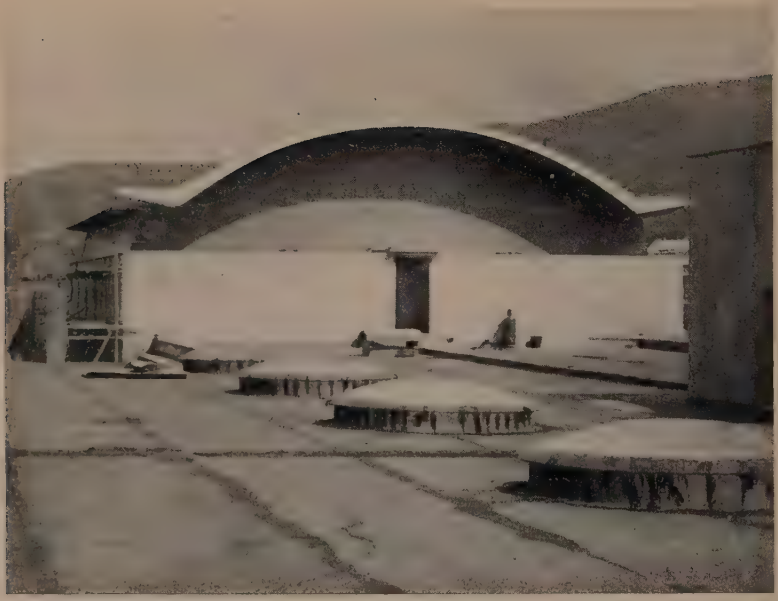
Reinforcement and Conduits in a Dome

Fig. 29



Roof of Main Production Area

Fig. 30



EXTERIOR OF EAST VAULT

Fig. 31

INTERIOR OF NORTHERN SECTION OF FACTORY

the boundary structures were designed to be adequate between the following limits :—

(1) Dome weight transferred entirely by shear (extensional case) with no live load on the walkway slabs. This gave the minimum compression in the arches and minimum tension in the ties.

(2) Eighty-five per cent of dome weight transmitted by shear and the remainder vertically with full live load on walkway slabs. This gave maximum compression in the arches and maximum tension in the ties.

For bending in the tie members the maximum of either case was taken.

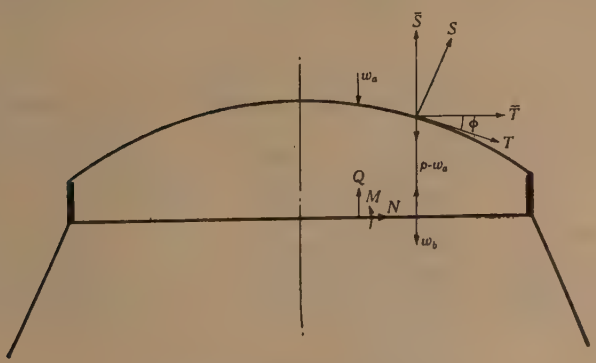
Figs 23, Plate 2, show the reinforcement details of the bow-string girder. The following analysis of the boundary structure has been included in pursuance of the Authors' aim to fill in the gap between structural theory and its practical result.

Design of Bowstring Girder

In this analysis of the bowstring girder forming the boundary structure to a longer side of the dome little general explanation of notation and method has been included; in the first place, they will be self-explanatory to many and, secondly, one of the Authors has presented to the Institution a Paper [7] giving a full explanation of the method of structural analysis here exemplified.

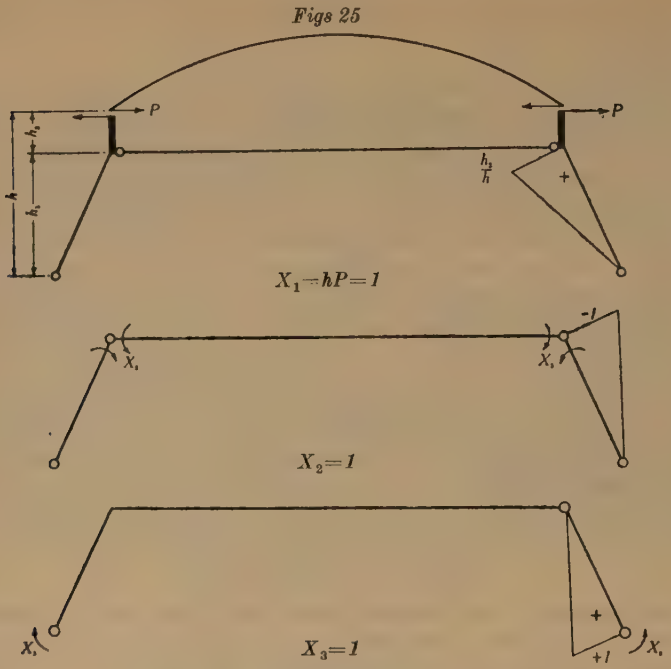
The redundants employed are shown in *Figs 25*. The forces in the members of the statically determinate structure (primary system) obtained

Fig. 24



Stress-Resultant Symbols

Figs 25



REDUNDANTS EMPLOYED

when these redundants are assumed to be zero are denoted by the superscript °.

Fig. 24 shows the symbols used for stress resultants.

Due to Loads

Arch.—In this primary system the arch takes all the tension, that is:

$$\bar{T}^\circ = -\left[\frac{\partial F}{\partial y}\right]_B$$

When p = tension in hangers per unit distance, the vertical shear in the arch is given by:

$$\bar{S}^\circ = \bar{T}^\circ \frac{\partial z}{\partial x} - \int_0^x \bar{T}^\circ \frac{\partial^2 z}{\partial x^2}\, dx + \int_0^x p^\circ dx$$

In the flexible arch the normal shear, $S = 0$

$\therefore$ the direct tension, $$T = \frac{1}{\cos\phi}\,\bar{T}$$

$$\text{and } \bar{S} = -\bar{T}\tan\phi = \bar{T}\frac{\partial z}{\partial x}$$

$$\therefore p = \bar{T}\frac{\partial^2 z}{\partial x^2}$$

Tie Beam.—If w_a denotes weight of arch rib per unit distance
w_b denotes weight of tie-beam, walkway, etc.
$w = w_a + w_b$

Then shear in tie-beam,

$$Q^\circ = \int_0^x w dx - \int_0^x p^\circ dx$$
$$= \int_0^x w dx - \int_0^x T^\circ \frac{\partial^2 z}{\partial x^2} dx$$

Bending in tie-beam, $M^\circ = \int_0^a Q^\circ dx - \int_0^x Q^\circ dx$

Tension in tie-beam, $N^\circ = -\frac{d}{h_1} W_t$

where W_t denotes the load at the foot of the column.

Due to Redundants

When $X_1 = Ph = 1$

Arch : $\bar{T} = -\frac{1}{h}$ and $T = -\frac{1}{h \cos \phi}$

Tie-beam : $Q = \frac{1}{h} \cdot \frac{\partial z}{\partial x}$; $M = -\frac{z}{h}$; $N = \frac{1}{h_1}$

Column bending : shown in *Figs 25* for all redundants.

When $X_2 = 1$

Tie-beam : $M = -1$; $N = -\frac{1}{h_1}$

When $X_3 = 1$

Tie-beam : $N = -\frac{1}{h_1}$

Influence Coefficients

It should be noted that integrating along the arc may be expressed in terms of integrating along x by $\int_0^s ds = \int_0^x \frac{dx}{\cos \phi}$

Integrating the loaded primary system with the redundant systems gives the relative displacements in the redundant directions :

$$U_1^\circ = -\frac{1}{hA_a}\int_0^a \frac{\bar{T}^\circ}{\cos^3 \phi} dx - \frac{1}{hI_b}\int_0^a zM^\circ dx - \frac{ad}{h_1^2 A_b} W_t$$

$$U_2^\circ = -\frac{1}{I_b}\int_0^a M^\circ dx + \frac{ad}{h_1^2 A_b} W_t$$

$$U_3^\circ = \frac{ad}{h_1^2 A_b} W_t$$

Integrating the redundant systems, two at a time, gives the relative displacements when each redundant in turn equals unity.

$$g_{11} = \frac{1}{h^2 A_a}\int_0^a \frac{dx}{\cos^3\phi} + \frac{1}{h^2 I_b}\int_0^a z^2 dx + \frac{a}{h_1^2 A_b} + \frac{l}{3I_c}\left(\frac{h_2}{h}\right)^2$$

$$g_{12} = \frac{1}{hI_b}\int_0^a z dx - \frac{a}{h_1^2 A_b} - \frac{l}{3I_c}\left(\frac{h_2}{h}\right)$$

$$g_{13} = -\frac{a}{h_1^2 A_b} + \frac{l}{6I_c}\left(\frac{h_2}{h}\right)$$

$$g_{22} = \frac{a}{I_b} + \frac{a}{h_1^2 A_b} + \frac{l}{3I_c}$$

$$g_{23} = \frac{a}{h_1^2 A_b} - \frac{l}{6I_c}$$

$$g_{33} = \frac{a}{h_1^2 A_b} + \frac{l}{3I_c}$$

The solution consists in finding the values of the redundants which eliminate these relative displacements in the primary system. This consists of solving the simultaneous equations, $GX + U^\circ = 0$. When this has been done, the total stress resultants in the members are as follows :

Arch : $$\bar{T} = \bar{T}^\circ - \frac{1}{h} X_1$$

Hangers : $$p - w_a = -\frac{1}{r\cos^3\phi}\bar{N} - w_a$$

Beam : $$Q = Q^\circ - \frac{1}{h} X_1 \tan\phi$$

$$M = M^\circ - \frac{z}{h} X_1 - X_2$$

$$N = N^\circ + \frac{1}{h_1} X_1 - \frac{1}{h_1} X_2 - \frac{1}{h_1} X_3$$

Column :

Bending moment at top $= \frac{h_2}{h} X_1 - X_2$

Bending moment at bottom $= X_3$

The numerical work now follows. In the integrations by Simpson's rule and the solution of the simultaneous equations for the redundants, all the numbers which were written down are given. This is evidence of the usefulness of a calculating machine in an engineer's office.

It is interesting to note that, although arch shortening and tie stretching were taken into account, these effects were negligible in this particular case.

From the results given, the composition of the reinforced-concrete members was determined in the ordinary way.

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The Paper is accompanied by seven photographs and twenty-seven sheets of drawings and diagrams, from which the Figures in the text and folding Plates 1 and 2 have been prepared, and by the following Appendix.

APPENDIX

The following Tables indicate the numerical work which is involved in the foregoing theory.

TABLE 1.—NUMERICAL

Point	0	1	2	3	4
					Integration from
Even points	1	4	1 1	4	1 1
Odd points Symmetric function	2	1 1	4	1 1	4
Asymmetric function	0	1·5 1	4	1 1	4
					Total integration
	1	4	2	4	2
			Long side		
x	0	4·1458	8·2917	12·4375	16·5833
z	8·2977	8·2178	7·9780	7·5772	7·0134
z^2	68·852	67·532	63·648	57·414	49·188
Tan $\phi = -\frac{\partial z}{\partial x}$	0	·03847	·07711	·11609	·15561
$\frac{1{,}000}{R\cos^3\phi} = -1{,}000\frac{\partial^2 z}{\partial x^2}$	9·2592	9·2798	9·3419	9·4470	9·5975
$1/\cos^3\phi$	1·00000	1·00222	1·00893	1·02028	1·03654
			CASE I		
From shell stresses:					
$\frac{\overline{T}^\circ}{1{,}000} = \int_0^x \frac{N_{xx}}{1{,}000}\,dy$	229	228	224	216	205
$-\overline{T}^\circ\frac{\partial^2 z}{\partial x^2}$	2,120	2,116	2,093	2,041	1,967
$-\int_0^x \overline{T}^\circ\frac{\partial^2 z}{\partial x^2}\,.\,dx$	0	8,800	17,520	26,090	34,400
$\int_0^x w\,.\,dx = wx$	0	4,975	9,950	14,925	19,900
Hence Q°	0	13,775	27,470	41,015	54,300
$10^{-3}\int_0^x Q^\circ dx$	0	28·5	114·1	256·1	453·8
$10^{-3}\,M^\circ$	2,685·8	2,657·3	2,571·7	2,429·7	2,232·0
$10^{-3}z_M{}^\circ$	22,286	21,837	20,517	18,410	15,654

INTEGRATION

5	6	7	8	9	10	
0 to x (accumulation)						
4	1		1	4	1	External multipliers :—
	1	4	1			Long side : $\frac{\Delta_x}{3} = \frac{4{\cdot}1458}{3}$
1		1	4	1		$= 1{\cdot}3819$
1	4	1				
1		1	4	1		Short side : $\frac{\Delta_y}{3} = \frac{3{\cdot}1875}{3}$
1	4	1				$= 1{\cdot}0625$
and check						
4	2	4	2	4	1	
		$R = 108$ feet				Units are lb. and feet unless otherwise stated.
20·7292	24·8750	29·0208	33·1666	37·3125	41·4583	
6·2842	5·3860	4·3145	3·0634	1·6289	0	$\int_0^a z dx = 231{\cdot}16$
39·491	29·009	18·615	9·384	2·653	0	$\int_0^a z^2 dx = 1539{\cdot}7$
·19585	·23703	·27938	·32317	·36872	·41638	
9·7970	10·0503	10·3641	10·7470	11·2103	11·7687	
1·05808	1·08544	1·11933	1·16068	1·21072	1·27103	$\int_0^a \frac{dx}{\cos^3\phi} = 44{\cdot}9081$
		CASE I				
190	171	146	114	72	0	
1,861	1,719	1,513	1,225	807	0	
42,350	49,790	56,510	62,240	66,505	68,380	$10^{-3} \int_0^a \frac{\overline{T^\circ}}{\cos^3\phi} dx$
24,875	29,850	34,825	39,800	44,775	49,750	$= 7385{\cdot}0$ where $w = 1{,}200$ lb./ft.
67,225	79,640	91,335	102,040	111,280	118,130	
705·8	1,010·5	1,365·2	1,766·5	2,209·2	2,685·8	
1,980·0	1,675·3	1,320·6	919·3	476·6	0	$\int_0^a \frac{M^\circ}{1{,}000} dx = 73{,}159$
12,443	9,023	5,698	2,816	776	0	$\int_0^a \frac{zM^\circ}{1{,}000} dx = 490{,}540$

TABLE 2.—INFLUENCE COEFFICIENTS

Coefficient	Arch shortening	Beam bending	Beam stretching	Column bending	Total
g_{11} . . .	0·188	2·802	0·081	0·045	3·116
g_{12} . . .	—	6·732	−0·081	−0·480	6·171
g_{13} . . .	—	—	−0·081	0·240	0·159
g_{22} . . .	—	19·319	0·081	5·125	24·525
g_{23} . . .	—	—	0·081	−2·562	−2·481
g_{33} . . .	—	—	0·081	5·125	5·206
			CASE I		
$10^{-3}\ U_1^\circ$ . .	−492	−14,287	−62	—	−14,841
$10^{-3}\ U_2^\circ$ . .	—	−34,092	62	—	−34,030
$10^{-3}\ U_3^\circ$ . .	—	—	62	—	62

Simultaneous equations $GX = -U^\circ$

$$3{\cdot}116X_1 + 6{\cdot}171X_2 + 0{\cdot}159X_3 = 14{,}841$$
$$6{\cdot}171X_1 + 24{\cdot}525X_2 - 2{\cdot}481X_3 = 34{,}030$$
$$0{\cdot}159X_1 - 2{\cdot}481X_2 + 5{\cdot}206X_3 = -62$$

Reciprocal of G to obtain solution by $X = -G^{-1}U^\circ$

	3·116	6·171	0·159
	0·3209	−1·9803	−0·0510
		24·525	−2·481
		12·305	−2·796
		0·08127	0·2272
			5·206
			4·563
			0·21915
G^{-1}	·69460	−·18588	−·10978
	−·18588	·09258	·04979
	−·10978	·04979	·21915
$-10^{-3}U^\circ$	14,841	34,030	−62
Hence solution			
$10^{-3}\{X_1\ X_2\ X_3\}$	3,990	389	52

Case I is completed in Tables 3 and 4 on pp. 36 and 37.

CASE II

This case is based on the assumption that tangential shell stresses account for 85 per cent of the weight of the dome. The remainder is accounted for by vertical loads :

$$\int_0^a \frac{1}{R\cos^3\phi}\,\bar{T}_B^\circ\ dx = 58{,}100 \text{ lb.}$$

corresponding to uniform $w_B = 1{,}750$ lb. per foot (including live load = 300)

$$\int_0^b \frac{1}{r\cos^3\phi}\,\bar{T}_A^\circ\ dy = 47{,}900 \text{ lb.}$$

corresponding to uniform $w_A = 1{,}500$ lb. per foot (including live load = 300).

Thus the total column load is the same as for Case I with the walkway live loads added, giving $W_l = 226{,}200$ lb.

The numerical solution is the same as for Case I with these modifications and is summarized in Tables 5 and 6.

TABLE 5.—INFLUENCE COEFFICIENTS. CASE II

Coefficient	Arch	Beam		Column	Total
		Bending	Stretching		
$10^{-3}\ U_1°$	−418	−15,514	−69	—	−16,001
$10^{-3}\ U_2°$	—	−37,056	69	—	−36,987
$10^{-3}\ U_3°$	—	—	69	—	69

With the same G as before, $X = -G^{-1}\ U°$ gives

$10^{-3}\{X_1\ X_2\ X_3\} = \{4{,}247\quad 447\quad 70\}$

TABLE 6.—SUMMARY OF STRESS RESULTANTS. CASE II

Point		0	1	2	3	4	5	6	7	8	9	10	
Arch	$10^{-3}\ T$	−70	−71	−75	−82	−92	−105	−123	−146	−177	−217	−287	1,000 lb.
Beam	$10^{-3}\ M$	264	254	227	181	118	42	−48	−147	−252	−354	−447	1,000 lb.-feet.
	$10^{-3}\ Q$	0	5	9	13	17	20	23	25	25	24	21	1,000 lb.
	N	198,700 lb.											
Hangers Net Tension ($w_d = 448$)		200	211	253	317	425	561	758	1,013	1,357	1,839	2,671	lb. per foot

TABLE 3.—STRESS

Point		0	1	2	3	4
Arch						
Compression	$10^{-3}\,\overline{T}^{\circ}$	229	228	224	216	205
	$-10^{-3}\,\frac{X_1}{h}$	−249	at all points			
	$10^{-3}\,\overline{T}$	−20	−21	−25	−33	−44
	$10^{-3}\,T$	−20	−21	−25	−33	−45
Stress		150	160	190	240	330
Beam						
Bending	$10^{-3}\,M^{\circ}$	2,686	2,657	2,572	2,430	2,232
	$-10^{-3}\,\frac{z}{h}\,X_1$	−2,066	−2,046	−1,987	−1,887	−1,746
	$-10^{-3}\,X_2$	−389	at all points			
	$10^{-3}\,M$	231	222	196	154	97
Shear	$10^{-3}\,Q^{\circ}$	0	14	27	41	54
	$-10^{-3}\,\frac{\tan\phi}{h}\,X_1$	0	−10	−19	−29	−39
	$10^{-3}\,Q$	0	4	8	12	15
Tension	N^{c}		−52,800	at all points		
	$\frac{1}{h_1}\,X_1$		275,200	,,	,,	
	$-\frac{1}{h_1}\,X_2$		−26,800	,,	,,	
	$-\frac{1}{h_1}\,X_3$		−3,600	,,	,,	
	N		192,000	lb. ,,	,,	
Hangers						
	$\overline{T}\,\frac{\partial^2 z}{\partial x^2}$	185	195	234	312	422
	$-w_a$	−200	at all points			
Net tension	$p - w_a$	−15	−5	34	112	222

TABLE 4.—SUMMARY OF SHORT SIDE

$\frac{1{,}000}{r\cos^3\phi} = -1{,}000\,\frac{\partial^2 z}{\partial y^2}$	12·1	12·1	12·2	12·3	12·5
$\frac{\overline{T}^{\circ}}{1{,}000} = -\int_0^A \frac{N_{yy}}{1{,}000}\,dx$	191	189	185	179	169
$-\overline{T}^{\circ}\,\frac{\partial^2 z}{\partial y^2}$	2,311	2,287	2,257	2,202	2,113

∴ Weight of quarter dome =

$$\int_0^a \frac{1}{R\cos^3\phi}\,\overline{T}_B^{\circ}\,dx + \int_0^l \frac{1}{r\cos^3\phi}\,\overline{T}_A^{\circ}\,dy = 68{,}400 + 56{,}400 = 124{,}800 \text{ lb.}$$

(compare with p. 367)

ESULTANTS. CASE I

5	6	7	8	9	10	
190	171	146	114	72	0	
−59	−78	−103	−135	−177	−249	
−60	−80	−107	−142	−190	−270	1,000 lb.
440	590	790	1,050	1,410	2,000	lb. per sq. in.
1,980	1,675	1,321	919	477	0	
1,565	−1,341	−1,074	−763	−406	0	
26	−55	−142	−233	−318	−389	1,000 lb.-feet.
67	80	91	102	111	118	
−49	−59	−70	−80	−92	−104	
18	21	21	22	19	14	1,000 lb.
578	784	1,068	1,451	1,984	2,930	
378	584	868	1,251	1,784	2,730	lb. per foot.

OR LOAD CHECK AND W_t ($r = 82{\cdot}75$ feet)

12·8	13·1	13·5	14·0	14·6		
156	140	119	92	58	0	
,997	1,834	1,607	1,288	847	0	$10^{-3}\int_0^b \frac{T^{\circ}}{\cos^3\phi}\,dy = 4664{\cdot}7$

Other loads :

On long side w_B = 1,200 lb. per foot. Therefore aw_B = 49,800 lb.

On short side w_A = 930 lb. per foot. Therefore bw_A = 29,600 lb.

Total vertical component of column load, W_t = 204,200 lb.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides and referred specially to *Figs 26* to *31* (between pp. 368 and 369).

Fig. 26 showed the storage basement under the main production area. In it could be seen the grid of holes through the flat slab floor, referred to on p. 347, and also the regular distribution of bolt sockets provided in the ceiling for receiving hangers to support any services that might be required. The columns shown were those referred to on pp. 11 and 12 (*Fig. 4*).

Fig. 27 showed the interior of the main production area and the underside of one of the dome structures, dealt with in the Paper from p. 361 onwards. The sixteen circular openings in a shell-concrete dome could be seen ; half of them were for natural lighting and half for the provision of housings, accessible from above, for fluorescent lighting.

Fig. 28 was a view from above of part of a dome before concreting. The electric wiring conduits to be buried in the concrete could be clearly seen.

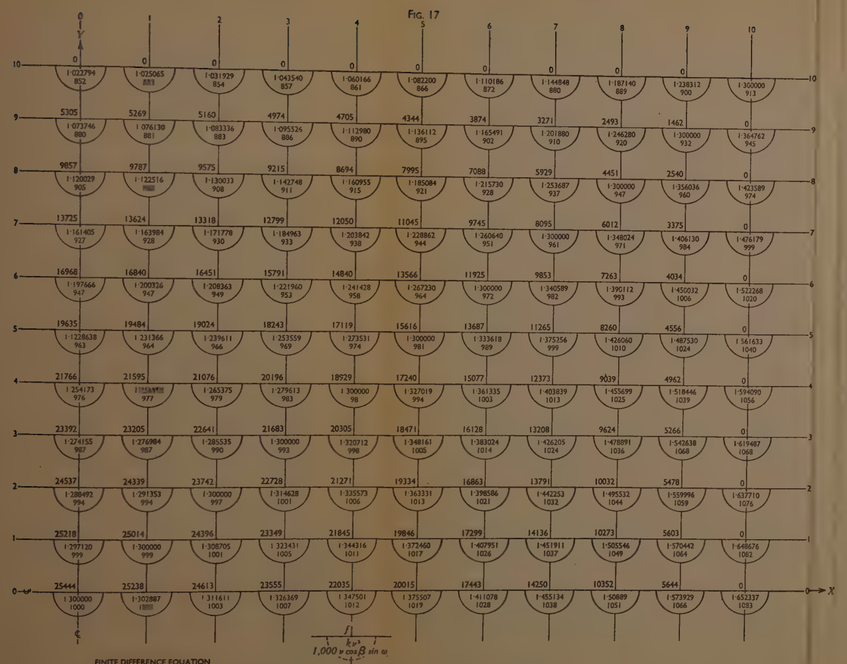
Fig. 29 showed the roof of the main production area and particularly the flat concrete slabs held up by the bow-string girders, referred to in the Paper as walkways. It had been explained on p. 368 that that proportion of vertical load to shell load went some way towards satisfying the boundary conditions assumed by the membrane theory. It could be seen from the photograph that a means of obtaining virtual structural separation of the domes for expansion and contraction was a considerable problem. The description on p. 357 et seq. and in *Figs 8* and *9* showed how the problem had been solved.

Fig. 30 was an end-on view of the east vault, referred to on p. 359, whose actual proportions could be gauged from the aerial view in *Fig. 6*. It showed that sagging of the edge had been avoided by the means described and illustrated in *Fig. 14*. That view provided a fairly sensitive record for future comparison for those who, like Mr Cousins, had expressed surprise that increased deflexion due to creep had not been detected in that instance.

Fig. 31 showed the interior of the northern section, described very briefly in the Paper, and it had been included to illustrate the slenderness of the structure above crane-beam level.

The Chairman, when proposing the vote of thanks to the Authors, remarked that *Fig. 6* made it clear that the Authors were dealing with a very unusual structure which would redound very much to the credit of British engineers and architects. He congratulated the Authors on bringing before the Institution the mathematical work upon which the design had been based. It required a certain amount of courage to present anything harder than simple algebra to the practical engineer, who did not really believe that mathematics would get him anywhere. In the present case, however, it could not be doubted that the structure was not amenable

FIG. 17

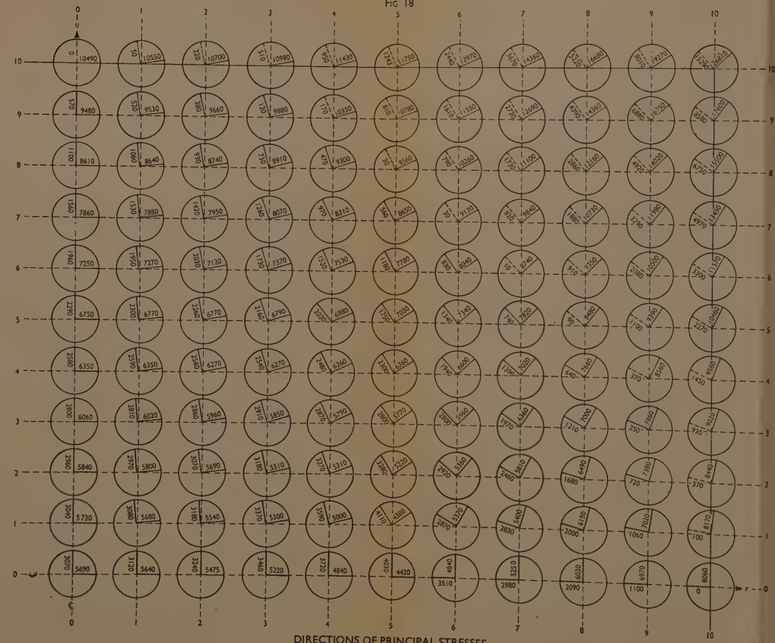


FINITE DIFFERENCE EQUATION

$$R \equiv (f_1 + f_3 - 2f_5) + k_2{}^2(f_2 + f_4 - 2f_5) + 1{,}000\, w \cos\beta \sin\omega$$

RELAXATION DIAGRAM FOR STRESS FUNCTION

FIG. 18



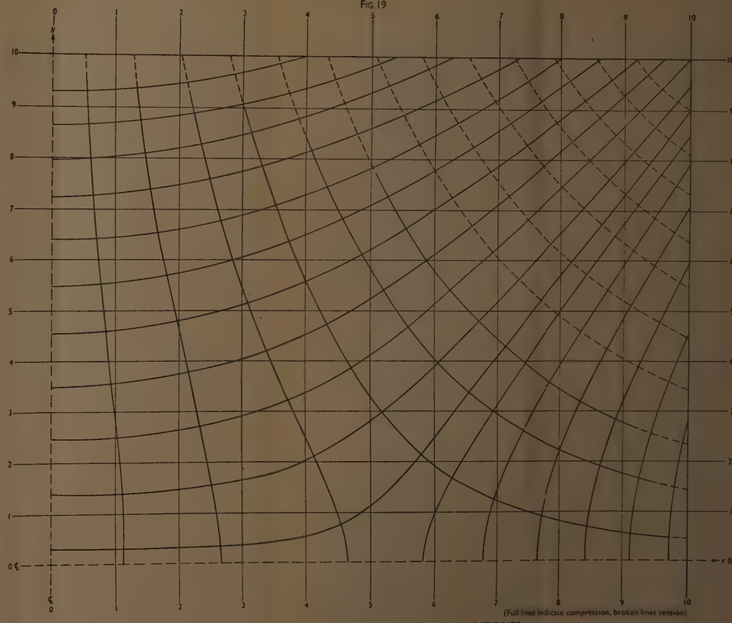
DIRECTIONS OF PRINCIPAL STRESSES

(Stresses in lb. per foot run on plan)

PLATE I

REINFORCED-CONCRETE FACTORY AT BRYNMAWR

FIG. 19



(Full lines indicate compression, broken lines tension)

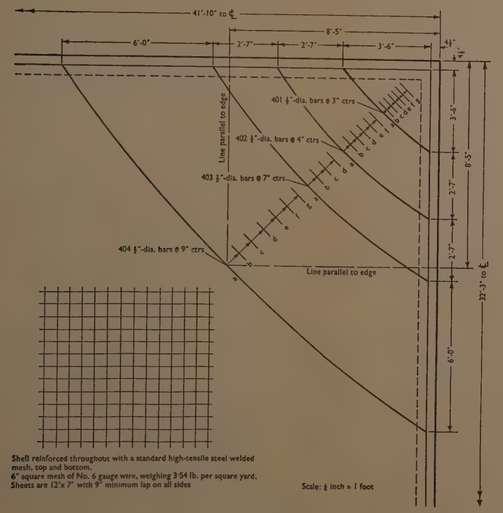
QUARTER PLAN OF SHELL SHOWING STRESS TRAJECTORIES

O. N. ARUP and R. S. JENKINS

WILLIAM CLOWES & SONS, LIMITED : LONDON

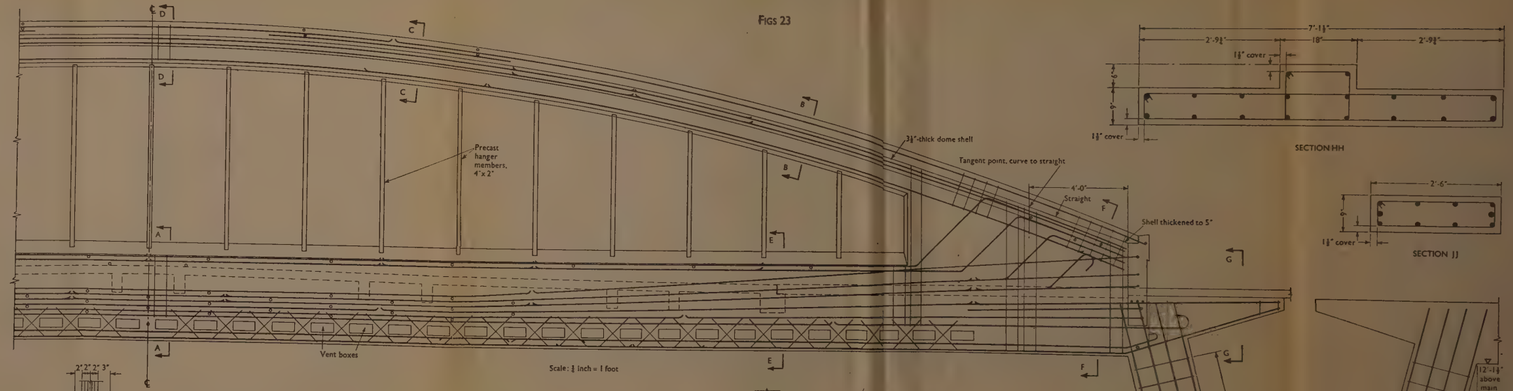
THE DESIGN OF A REINFORCED-CONCRETE FACTORY AT BRYNMAWR, SOUTH WALES

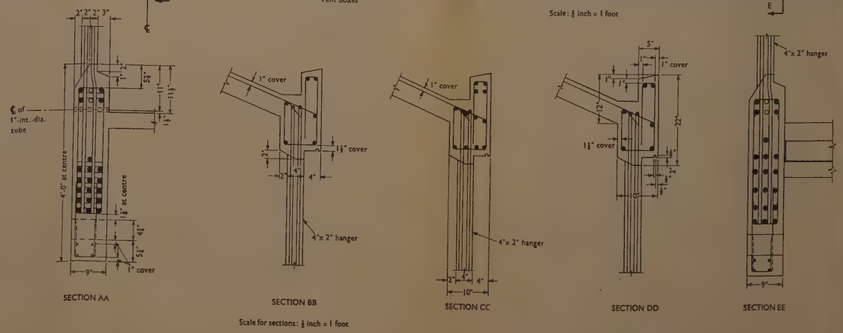
FIG. 20

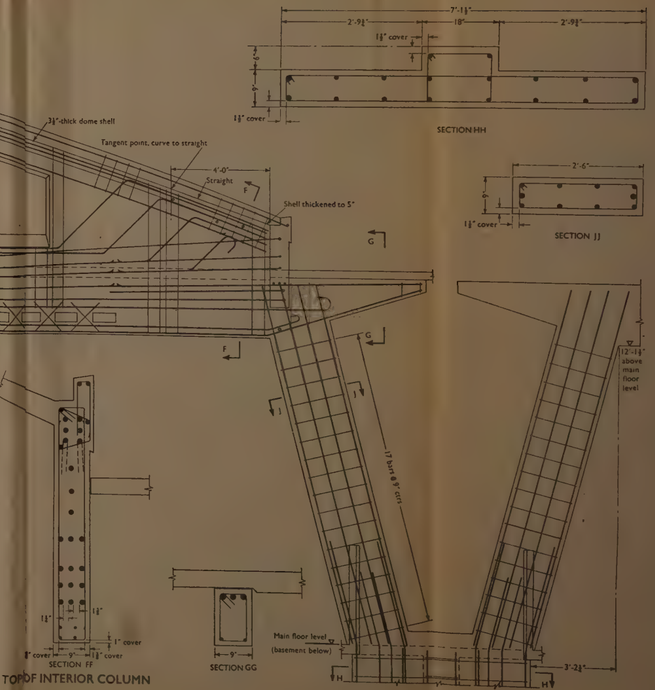

41'-10" to ℄
6'-0"
2'-7"
2'-7"
3'-6"
8'-5"
4½"
401 ½"-dia. bars @ 3" ctrs
402 ½"-dia. bars @ 4" ctrs
403 ½"-dia. bars @ 7" ctrs
404 ½"-dia. bars @ 9" ctrs
Line parallel to edge
Line parallel to edge
3'-6"
8'-5"
2'-7"
2'-7"
6'-0"
32'-3" to ℄
Shell reinforced throughout with a standard high-tensile steel welded mesh, top and bottom.
6" square mesh of No. 6 gauge wire, weighing 3·54 lb. per square yard.
Sheets are 12'x 7' with 9" minimum lap on all sides
Scale: ½ inch = 1 foot

RELATION BETWEEN STRESS LINES AND REINFORCEMENT AT CORNER OF SHELL

FIGS 23







REINFORCEMENT DETAIL OF LONG-SPAN DOME GIRDER AND TOP OF INTERIOR COLUMN

O. N. ARUP and R. S. JENKINS

The Institution of Civil Engineers

WILLIAM CLOWES & SONS

to any ordinary simplifications. It was not possible to write down the bending moment in foot-tons and then turn to a convenient table and find the required section modulus.

The Authors had produced a very beautiful structure, and it should be emphasized that that was the justification for the mathematics. In case anyone should feel inclined to say " All this mathematics is not engineering," it should be pointed out that it very definitely was engineering and that without it the structure described in the Paper would not have been built. The Chairman felt strongly that the work described was a magnificent example of how mathematics could be interpreted in terms of real engineering construction.

Mr H. G. Cousins observed that the method of filling the ground cavities with colloidal grout seemed to be very effective, but there was one point about which he was not clear. The quantity of grout used was stated to be 15 per cent of the total volume of the Yard Vein. Had it been considered that the remaining 85 per cent of the volume had been filled by subsidence ?

It would be interesting to know whether any troubles had been experienced with the sliding joints supporting the cruciform slabs between the domes. He had always avoided such sliding joints, preferring definite gaps to take care of temperature movements. The method of waterproofing those joints would also be of interest. He noticed that more normal joints with double columns had been used on the western vaults, and he wondered whether those joints had gone right along the edge beam. He had frequently used joints which went only part of the way along the edge beam, relying on the flexibility of the vault in the centre.

The joint in the northern section was certainly novel, and he did not like it very much, especially since it seemed to involve additional longitudinal stiffening beams. He wondered whether it would not have been sufficient to hinge the end frame, and whether that frame was in fact split, as the Paper stated, because if so there could be no tension in that frame to balance the compression along the crown, which had to be supplied by bending of the columns. The east vault was a very good example of the need for radial stiffness at an edge unless the deflexions were to become large. He was rather surprised, however, that increase of deflexion due to creep could not be detected.

Coming to the design of the dome roofs, he would first of all like to express his gratitude to Mr Jenkins for the work which he had carried out on the design of shells, first in his publication on cylindrical shells [8] and then in his Paper [9] on doubly-curved shells in the recent Symposium, and finally in the present complementary Paper giving details of the particular

[8] R. S. Jenkins, " Theory and Design of Cylindrical Shell Structures." Lund Humphries & Co. Ltd.

[9] R. S. Jenkins, " The Theory of New Forms of Shell." Symposium on Concrete Shell Roof Construction. C.A.C.A., 1953.

example. It was always an intellectual treat to delve into his mathematical analyses, even though it might prove a little exacting at times to lesser mortals.

It was with considerable trepidation, therefore, that Mr Cousins ventured to discuss the mathematical aspect of the subject, and he would confine himself to the easy task of criticizing the assumptions. The chief assumption in that case was that of no outward force at the boundary. The main problem in shell analysis was the solution of the effects at boundaries. The assumption of no outward force was a particular case of disturbance at an edge and was equivalent to applying an outward force to a dome otherwise in compression and was a very limiting assumption. Had the bow-string girders been sloping instead of vertical, which might have improved the lighting, the inaccuracy would begin to become large using that method. Also, since the slope to the horizontal at the edge was very small, a small normal force involving bending could combine with a much larger tangential force without exerting horizontal thrust at the girders.

Mr Jenkins had touched on the question of discontinuity at the edge, and had indicated[10] an approximate equation of compatibility which might be limited to rapid damping. It would be of great help if he would extend at the same time his analysis, as he had done for cylindrical shells, so that the analysis would cover all conditions at the boundary surface. Mr Cousins knew that he was asking for a great deal there, but he was asking it of the right person.

He did not suggest that there was any fundamental inaccuracy in the design, either in the tension steel required in the shell or in that in the girders. The Authors, he thought, had been very wise to base the design of the girders on the total tension to be expected, since that would not be greatly affected whatever the distribution in the shell itself. It was noticeable in the design that the compression in the dome was a minimum at the crown, increasing towards the edge and parallel to the edge. Had that same edge beam been used with a plain cylindrical shell, the reverse would have been the case. It seemed a little strange, therefore, that a cylindrical shell could have a more favourable lever arm than the doubly-curved one. The smaller lever arm taken in the present case, was of course, on the safe side.

Mr P. G. Bowie said that it was obvious that in applying a very delicate mathematical analysis to a structure of the kind in question it was of special importance to take trouble to ensure that the premises were correct. The premise of which one would naturally think first was the avoidance of any settlement which might otherwise upset the equilibrium of the structure. It was very plain how thoroughly the Authors had investigated that question and had guarded against any possible failure in that respect.

[10] *Ibid.*

Mr Bowie was particularly interested in the Authors' use of bored piles resting directly on shale and probably giving a bearing stress beneath them of 400 to 500 lb. per square inch. He was sure that they were quite satisfactory, but he wondered whether there would have been any great objection to moving the column bodily so as to be over the pile. When dealing with a contract, however, there had to be certain limits to the extent to which the contractor could be allowed to depart from the drawings.

Would the Authors say a little more about the foundations given to the groups of inclined columns supporting the domes? They were not specifically referred to, but presumably they were of a group-pile form. He imagined that the load on them was 400–500 tons.

Despite the care given to ensuring that there should be no settlement, very great care had been given to the jointing of the centre portion. In that connexion it would be most interesting to know what precautions, if any, had been taken to ensure that the jointing was waterproof. That was the main difficulty, he imagined, with such a roof, especially when, owing to its form of intersecting channels, it was particularly liable to collect snow, with consequent flooding of any gutters which might be there.

Mr C. B. Brown confined his remarks to the site difficulties at Brynmawr. They had particularly attracted his attention in view of the fact that his firm had had something to do during the past year with a job which involved serious mining subsidence problems. It was an existing factory which had been very badly damaged in the past, and looked like being very much more damaged in the future.

The mine workings concerned were about ten seams, ranging in thickness from 3 feet to 6 feet and at depths ranging from 1,000 to 2,500 feet. The long-wall system of mining had been used since about 1919. Admittedly it had been very difficult to assess exactly what subsidence had taken place as the result of the collapse of workings in the past. It was also very difficult to forecast future settlement which might result, owing mainly to the very irregular nature of the ground due to extensive faulting. What they had managed to conclude with a certain degree of certainty, however, was that, owing to the collapse of a seam 4 feet 6 inches in thickness, about 3 feet of settlement had occurred between 1937 and 1940. That seam was at a depth of about 2,300 feet.

The factory concerned was a two-storey structure of rigid reinforced-concrete-frame construction, about 400 feet long by 150 feet wide, and was placed just on the edge of the mining area with its longitudinal axis running right across the area. The result of that had been that each of the successive seams which had been worked in the past sent a wave of subsidence across the ground towards the factory, the subsidence tending to peter out at about the spot where the factory was situated. The result of that on a rigid reinforced-concrete frame could well be imagined. The present position was that the west end of the factory was 4 feet 6 inches

lower than the east end, and there were numerous cracks all over the structure. That was not the end of the story, because mining operations were still going on and, if the Coal Board carried out their programme, it looked as though that unfortunate building was going to have a very uneasy time in the near future.

The point of mentioning all that was that their observations on the effects of seams at depths of about 2,000 feet appeared to indicate that depth was not quite such an important factor as might be supposed from the Paper; and that however deep the seam was, subsidence was almost bound to take place to a greater or lesser extent. In view of that fact, he would like to have some more information about the Old Coal Vein which was situated at a depth of 130 feet. Numerous borings had been taken on the site, and he wondered why at least some of those borings had not been taken down to a greater depth, in order to try to ascertain the actual conditions in those old coal workings.

The second point was that distribution of the workings over the whole area, as was mentioned in the Paper, did not appear to him to be of very great consequence in minimizing the risk of differential subsidence. The pillar-and-stall system had been used, and as the Authors rightly said, the results of the pillar-and-stall system were somewhat uncertain. It seemed to him to be quite possible that, owing to erosion of the pillars or some other cause, settlement might take place, but it was most unlikely that that settlement would take place simultaneously over the whole area. It appeared to be much more probable that settlement would take place first over a small area; and, if that small area were situated in a certain position, it might easily transfer differential subsidence to the surface, at the site of the factory. He would be very pleased if the Authors could give some more information about the investigations which had been made and the considerations which had led to the decision to disregard those old coal workings.

Mr E. O. Measor, with apology for what might seem to be a somewhat carping criticism but which was, at the same time, a tribute to the importance of the work in question, said that it would have been a great advantage if the Paper had been an authentic description of what was an extremely important structure, instead of referring the reader to descriptions in the technical Press, which seemed to be a very retrograde practice. The work was amongst those of which a description ought to appear in the records of the Institution. One result of the lack of a general description in the Paper was that the Paper itself was a rather disjointed miscellany of information, ranging from the grouting of mining cavities to a mathematical treatise bristling with differential expressions of the kind which members had learned to expect and value from Mr Jenkins.

Mr Measor had been surprised to read the comment on p. 348 (if he had understood it aright) that the Meadow Vein could be ignored because it had not been worked in the area concerned. He would have thought that

the fact that it had not been worked made it all the more important, unless the Coal Board had given an undertaking that they did not intend to work the seam. From a case of the same kind with which his own firm had been concerned in 1952, the Coal Board, if they had given any such undertaking, had acted differently from their usual practice, because they seemed to be very chary about saying whether they were going to work a seam in a particular place at any particular period. Would the Authors say something further about that ?

On p. 347 the Authors had referred to the selection of structural forms, but he felt that he should say, with due deference to them, that the reference seemed to be in the nature of a smoke-screen. Incidentally, their implied suggestion that the selection of economical structural forms was not usually deliberate and quantitative was not very fair to the structural engineering profession, because it should be both deliberate and quantitative. He wondered whether in the present case it had been any more so than in most. A study of the comments in the present Paper, and also in the *Architectural Review* for March 1952, to which the Authors had referred, suggested that there might be something in the fact that the building dated from a period when structural engineering and architecture tended to exaggerate the utility of shell-roof construction. The comment on p. 347 on the subject of narrow cylindrical shells might perhaps refer only to the shape of the shells and not to the use of shells as such. He would have thought that shells were not the solution to the problem of that part of the roof.

In the *Architectural Review* the architects had said, with regard to the main production area, " This had to have as large a portion as possible of its 77,000 superficial feet unbroken by structural supports. Accordingly, it is covered with nine domes, the largest of their kind in the world." Mr Measor thought that the Authors had done a great service to engineering by choosing to build those large domes of record dimensions, but he questioned whether the selection of them could be entirely justified either on the ground of economics or on the ground of the aesthetic well-being of the workers in the factory. The final selection usually depended to some extent on personal choice. Would the Authors say what alternative forms had been considered ?

The slides shown by the Authors when introducing the Paper had given Mr Measor a rather unfavourable impression of the natural lighting. The circular lights had appeared as holes in a rather dark ceiling, and he thought that that was attributable to the shape of dome used. A north-light shell usually led to a light ceiling. Would not other forms of construction, such as north-light shells or a roof with lantern lights, have given an equally clean appearance and a more favourable distribution of natural lighting ?

Mr Arup had referred in passing to the white spar on the waterproofing of the dome. Mr Measor felt that the addition of white spar made it more

difficult to inspect the waterproofing and to find out where the trouble was, if any trouble arose. Personally, he was opposed to its use.

He was interested in the Authors' description of the columns founded on the single piles and passing through what seemed to be a ground-bearing floor slab. The trouble with eccentricity arose, he assumed, from the fact that it was a ground-bearing floor slab and could not be incorporated with the head of the column, in which case it would no doubt have been able to absorb any eccentricity which occurred. That, he assumed, was a measure of economy, but it did call for extremely accurate pile-driving. A maximum error of 2 inches was very tight, and he wondered whether there was any indication that that had had an effect on the pricing of the piling.

The shell calculations had been carried out by the method of relaxation, and the Authors had seemed to imply that that was a suitable method for practical design. From experience, Mr Measor thought that it was not a method of practical design and that it cost an enormous amount of time and labour, work which could be done only by staff who were well qualified to be much better occupied. There was, he thought, an urgent need to develop modern methods for eliminating that whenever possible, and they could probably be applied to the kind of dome in question.

On p. 366 there was an explanation of the simplifications used for the calculations to which Mr Cousins had already referred. Mr Measor felt very strongly that, especially with reinforced or prestressed concrete, simplifications were essential, and any contribution which Mr Jenkins could make towards simplification was very important. Mr Measor would like to emphasize the importance of reliable approximations. He thought that there had been a tendency towards unnecessary accuracy in calculations for reinforced concrete. That was illogical, because he doubted very much whether it was possible to calculate the ultimate strength of reinforced-concrete members to an accuracy better than 15 per cent. He thought that there was a definite necessity for having a measure of agreement on the accuracy at which they should really aim. Probably 5 per cent would be ample for reinforced concrete. The section of the Paper in which Mr Jenkins dealt with the simplifications was very valuable.

The bow-string girder shown in *Fig. 22* had been treated in an idealized form with an end bracket with eccentricity h_2. In fact, if he understood Figs 23, Plate 2 correctly, the load distribution along the tie was such that it virtually eliminated that eccentricity where the column thrust and the arch thrust and the tie met.

Dr P. B. Morice gave an assurance to those uncertain about the quality of the natural lighting that, standing within the structure, one gained a striking effect of freedom, of lightness, and of a great enclosed area with a roof which seemed to be supported at very few points.

He would like to ask the Authors what had governed the choice of a membrane thickness of $3\frac{1}{2}$ inches compared with the normal value of about $2\frac{1}{2}$ inches. That seemed important since dead weight had determined the

size and included steel of the boundary members. The buckling loads of doubly curved shells were very large and he would be surprised if stability requirements were the reason.

Had any measurements of strain or deflexion of the boundary members been taken at the time of striking ? On a new form of structure, such as the Authors had described, it would be interesting to see comparisons between calculated and measured effects. He thought that the Institution was seeking such information for the Congress to be held in 1955.

He had been interested in the cruciform slabs as a solution to temperature movements of the main structure. Had any insulation been provided on the inside or outside of the main domes ? He gathered that, on the whole, external insulation was preferred nowadays to reduce temperature variation of the concrete membrane itself.

Small structural models of non-developable surfaces were difficult to make, except in concrete or plaster, and were not convenient for office use. He had recently been experimenting with a small concrete model, 4 feet square and $\frac{1}{4}$ inch thick, of anticlastic form. Initial loading up to 60 lb. square foot had produced very small membrane stresses. An attempt had been made to carry out a calculation following the general theory suggested by Mr Jenkins,[11] but relaxation of the governing hyperbolic equation was unsatisfactory owing to inadequate convergence. He hoped Mr Jenkins might comment upon that.

Mr M. A. R. Powers added his assurance that the ceilings in the domes were in fact remarkably light. The slides which the Authors had shown might have been misleading on that point. It rained a good deal in Wales—at Brynmawr more than elsewhere—and it was very often dull out-of-doors ; but on many hundreds of visits which he had paid to the building during the course of its construction and after the domes had been put up and finished on the underside, it had often seemed to him that it was lighter indoors than out-of-doors. That could not really be so, but the building did give that feeling of extraordinary lightness.

There were one or two other questions which perhaps he might answer. The white Derbyshire spar on the outside of the roof had been put there for the purpose of insulation. He was afraid that it had not been a great success. They had always feared that it might come off, and in fact it was doing so. Had it been a true spar it might have stayed on, because that was fairly sharp, but marble chippings had been used instead, which were inclined to be round, and which had very little grip on the roof and so tended to come off. Some anxiety had been felt about using spar, because, being so sharp, it might puncture the felt.

The insulation on both the barrels and the domes was $\frac{1}{2}$-inch sprayed asbestos on the underside, which was finished with an asbestos spray paint.

Mr Hugh Tottenham observed that the form of shell which the

[11] See reference 1, p. 373.

Authors had taken was a circular arc translating on another circular arc. The calculations could be considerably simplified by using an elliptical paraboloid (a parabola translating along another parabola). It might be thought that that would give a different appearance, but the surface was within $\frac{1}{4}$ inch of the surface as shown, so that he did not think that the difference would be noticeable.

It would also simplify the calculations if, on p. 362, $Z\sqrt{g}$ could be taken as a constant, which meant assuming that the load was not quite uniform but was falling off towards the edge (alternatively, an allowance could be made for that by using a Fourier series for the load). The equation:

$$Z = \frac{x^2}{2R_1} + \frac{y^2}{2R_2}$$

represented an elliptical paraboloid and on that basis the equation on p. 362 would give directly:

$$\frac{1}{R_2}\frac{\partial^2 F}{\partial x^2} + \frac{1}{R_1}\frac{\partial^2 F}{\partial y^2} + c = 0$$

c being a constant. That equation could be solved directly by using a Fourier series in one variable for c. If the load were assumed to be increasing slightly towards the edge the equation would still admit solution in the same manner.

Another point which helped considerably arose from the fact that at the edge of the shell the loads were not carried by the shell but by the edge member. If that were taken into account a very much more convergent series would be obtained. Using those methods, Mr Tottenham had checked the Table of the forces along the centre-line and had obtained very close agreement. It was a pity that the Authors had not given the remaining forces in terms of N_{xx}, N_{xy}, and N_{yy}, because without them it was rather difficult to compare different approximations. Perhaps Mr Jenkins had already tried all those suggestions and might have some comments to make on them.

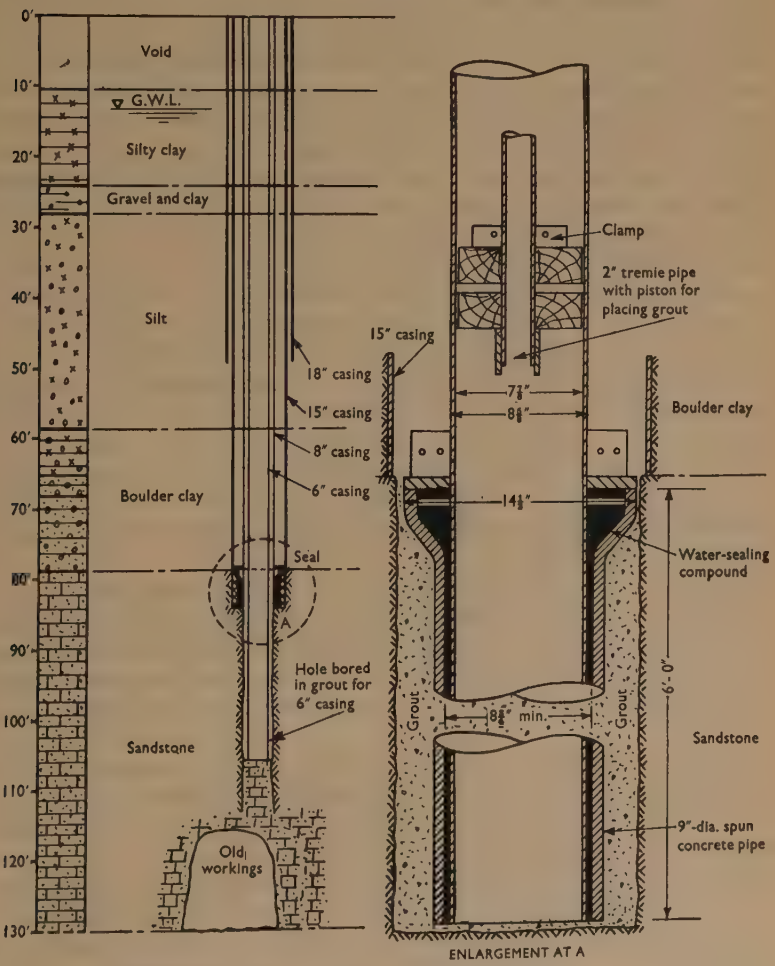
The Authors might be interested in the description of a test on a one-third-scale model of a similar type of structure carried out by Dr Hruban at Brno University. It had been loaded to 324 per cent of its design load and there had been no measureable overstrain in the shell, but it had been supported on four columns, and those had collapsed under the load. That was very reassuring for such a structure.

Mr M. W. Leonard said that the problem of constructing new works over areas subject to mining subsidence always called for a courageous approach, and the problems arising for the designer could be simplified if there was always adequate information available on the soil profile and water conditions, and if samples could be taken within the old workings. Such investigations provided their own problems, and it might be of interest to refer to a site investigation recently carried out in Scotland.

The problem had been to build a heavy chimney on a site in which

there was 60 feet of brown-grey alluvial silt overlying 20 feet of boulder clay which, in turn, was over sandstone in which there were old coal-mine workings ; it had been considered desirable to found on the sandstone, and

Figs 32



A Site Investigation in Scotland

that would probably have involved the sinking of a cylinder under compressed air ; it had therefore been essential to determine whether there was any difference in water conditions in the sandstone and the overlying silt, which were separated by the relatively impermeable boulder clay, and it

would have been very valuable information if the nature of the materials in the old workings could be determined.

From *Figs 32* it might be seen that the site investigation had been undertaken by sinking an 18-inch-diameter boring to 50 feet below ground level, and then continuing in 15-inch casing tube through the boulder clay until the sandstone had been reached ; careful readings had been taken of the water levels during the various stages of the boring. An unlined hole, approximately 14½ inches diameter, had then been drilled, using a percussion drilling rig, about 6 feet into the sandstone, and a water seal had been made in the top of the rock, as indicated in the sketch, using a 9-inch concrete pipe coated with a water-sealing agent. The concrete sleeve had been lowered inside the 15-inch boring on an 8-inch casing tube until it had been placed in the top 6 feet of sandstone. Grout had then been tremied into the annular space around the sleeve and a plug formed in the bottom of the 8-inch casing.

After a suitable waiting period the water in the 8-inch casing had been removed, and observations taken on the water within to check that the 8-inch pipe was watertight. The grout plug had then been broken out and immediately water had risen within the 8-inch casing to 12 feet below ground level. Thus the nature of the water conditions in the underlying sandstone had been determined, which would materially affect any construction which took place in that material.

Boring had continued into the sandstone to enable an examination to be undertaken of the materials within the workings by taking samples through a 6-inch casing.

It had been considered necessary that the old workings would have to be backfilled, and several schemes had been considered, such as grouting up the voids, drilling large-diameter holes from the surface, and filling them with concrete to form pillars of concrete which would transmit the load of the chimney from the top of the sandstone to below that which had been mined.

Mr Leonard noted from the Paper that a 19-inch-diameter bored pile, founding in the shale, was expected to carry a load of 60 tons, and he would be glad to know whether any of the piles had been test-loaded, and if so, what had been the load/time/settlement results.

Since the pile was presumably largely end-bearing, the loading conditions indicated that the shale would have to carry a permissible bearing load of about 32 tons per square foot, and that figure would appear to be high, unless the Authors had depended on some frictional value being obtained on the material around the walls of the pile ; if so, what was their estimate of that value.

Mr D. H. Little asked if the Authors would state the overall cost of the work.

Dr A. W. Skempton remarked that, although he knew very little about the technical details of the building at Brynmawr, he wished to

comment upon the question of economics which had been raised—particularly that by the last speaker. In Dr Skempton's opinion the architects and engineers had had the opportunity at Brynmawr to produce a work of art. To consider that there was, in the design of a building, more than simply efficiency and economics might be an old-fashioned viewpoint, but to make a building a work of art as well seemed to him to be an eminently worth-while objective. Whether, in fact, what had been done cost a little more than if some other method of construction had been used he did not know, and Dr Skempton did not see how anybody could prove the true value of the building solely by financial considerations. The Brynmawr factory was a structure which, from the technical point of view, was quite outstanding, and which was likely to enhance the reputation of British engineering. It was also a work which was self-evidently a fine piece of architecture. He would therefore take the liberty of suggesting (because he thought that the Authors might be diffident to do so) that Mr Little's question was slightly out of place. Dr Skempton thought that everybody should be pleased that, from time to time, work was produced of which they could be proud. He certainly felt proud to be a member of a profession which could produce such a work.

Mr W. P. S. Cockle, referring to what the previous speaker had said, observed that he had recently read a Paper [12] on the construction of the Marignane airport. The motive inspiring the French designers was noteworthy. The 1939 roof had been of 80 metres span, but they were now required to produce a roof spanning 100 metres. They said that their studies showed that the principle of design adopted for 80 metres was valid up to a limit of 100 metres; but they had noticed that successive improvements in the quality of high-tensile steel had led to the adoption of successively greater spans and gave scope for effecting economies, and they also thought that that would have been a lazy solution. They had therefore undertaken the research for a design for a span of 100 metres which would still be valid and even more economical for a span of 150 metres. The whole conception of that outstanding structure was, therefore, born of the need for the economical use of materials.

Mr Cockle would therefore stress that the cost of works was a consideration which should never be absent from the minds of members of the Institution.

*** **Professor A. A. L. Baker** remarked that Brynmawr factory was already deservedly famous as an achievement in industrial architecture, but to the reinforced-concrete designer the Paper conveyed a great deal more. It was a demonstration of how new, attractive, and efficient

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[12] N. Esquillan, "*Hangar à deux nefs de 101·5 mètres de portée de l'aéroport de Marignane*" ("Hangar of two 101·5-metre-span bays for Marignane Airport"). Ann. Inst. Bâtim., vol. 100, p. 822 (Sept. 1952).

*** This contribution was received in writing upon the closure of the oral discussion.—SEC. I.C.E.

architectural forms could be created by courageous designers who knew how to shape their material in three dimensions and how to determine shell stresses with the aid of modern mathematical techniques. The whole factory, from foundations to roof, was composed of structural forms logically related in a sound engineering manner to the support, loading, and general operational requirements. The most interesting of the various designs was probably the double-curvature shell roofs supported through tangential shear at the edges by tied flexible arches. The lines of principal stresses in the shells obtained by relaxation for vertical load, which was transmitted to the arches without outward radical pressure at the edges, were particularly interesting. They suggested roughly the action of a thin dome of about 40 feet diameter supported at the perimeter by a broad rectangular frame with deep corner haunches.* It was, however, not easy to visualize how tangential forces were transmitted to the arch at the edges when the adjacent principle stresses in the shell were almost normal or parallel to the edges. Since that was an important feature of the design, would the Authors add a little to their explanations ?

Although the Paper was mainly concerned with the structural aspects of design, most engineers would be interested to know something about costs. Could the Authors indicate whether such large-span double-curvature shell roofs, when repeated several times, were likely to compare reasonably in cost with other more common forms of construction ? Obviously, any such comparison should include long-term considerations such as low maintenance cost and the economic advantage of producing rubber more efficiently in pleasant, well-lit, and well-ventilated buildings. The Brynmawr factory was a dignified civilized building and its design would certainly have some prestige and advertisement value. It was a pity that the Authors had not included in the Paper an interior view in order to show the marked contrast with typical interiors of the more usual semi-permanent factory sheds.† In a recent radio broadcast it had been stated that in the United States the most modern factories were windowless and air-conditioned, and had high roofs supported by structural steelwork designed to carry heavy loads such as cranes and services. They were permanent efficiently designed structures, which could be used when required for various kinds of production. Brynmawr seemed to have achieved the same purpose, but with good roof lighting and some impressive architectural effects. The initial cost might have been high by more common standards, but in the long run would doubtless be justified, for there was something fundamentally unsound and, eventually, uneconomic in carrying on important production in drab utilitarian semi-permanent sheds.

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* Since the preparation of this contribution, it had been pointed out to Professor Baker that the dotted edges of the diagrams were centre-lines of the shell across which normal principal stresses would, of course, be expected.

† A number of the slides shown by the Authors when introducing the Paper are reproduced as *Figs 26* to *31*, between pp. 368 and 369.

Mr Ove Arup, in reply, said that the reason why the Authors had not given a proper description of the whole work instead of referring to articles in the technical Press, was that a Paper presented to the Institution should contain only original work. They had therefore been anxious not to repeat what had already been published elsewhere. That incidentally had the advantage of enabling them to concentrate more on matters which would be of interest to engineers.

There had been various questions and some interesting contributions on the problem of foundations in areas where subsidence attributable to old mine workings might be expected. That was exactly what had been hoped for when the Authors decided to describe the methods used by them in that instance, because it was a problem of general interest to engineers. Unfortunately, the records kept of that work were not sufficient to answer all the questions raised, and since a long time had elapsed since the work had been done, the details had slipped from the memory ; the main facts were, however, given in the Paper. The old workings had been divided into those which for various reasons were considered safe, and those which might still contain dangerous cavities ; the latter were grouted in. The quantity of grout used suggested that there was only 15 per cent of the original cavity left, calculated on the total volume of the vein. The rest had presumably been taken up by subsidence and by the pillars and possibly debris left in the workings, but in what proportion was unknown.

The seams that had not yet been worked were not dealt with at all. Coal mining had long ago been abandoned in the area, and although no guarantee had been received that the coal might not be extracted at some future date, it seemed very unlikely that that should happen. It was one of those risks which had to be taken, for there was no safeguard against it. In the extremely unlikely event that further extraction of coal would be an economic proposition in the future, the fact that the factory was in existence would have to be taken into account.

On the question of the eccentricity of the piles, it was correct that the problem arose from the fact that the basement floor slab was not suspended from the piles, but rested on the ground. It could have been dealt with by placing the columns on top of the piles, as suggested by Mr Bowie, but it would have been unsatisfactory and unsightly to allow the columns to be so much out of line, and it would have created complications in the formwork and the reinforcement of the main floor slab. It was not difficult to obtain the specified accuracy in the placing of the piles ; very few did not comply with the specification, and the cost of piling was hardly affected.

The main domes were supported at the four corners by groups of similar piles, as Mr Bowie had surmised. 60 tons was a normal load for a 19-inch bored pile. The boring provided a fair amount of information about the ground penetrated, and at Brynmawr the shale was very hard to bore into. Test loading had been carried out to the Authors' satisfaction.

On the question of the cruciform expansion joints provided over the main production area, the arrangement chosen was the most economical that the Authors could devise, and it had reduced the length of joints and the extra reinforcement to the minimum. The provision of definite gaps all round the domes would have been much more expensive. The joints had worked satisfactorily, and no leakages had occurred so far as the Authors knew.

The questions of lighting and insulation had been dealt with by Mr Powers. Mr Measor had suggested that the Authors' reference to the selection of the structural forms was in the nature of a smoke-screen. That reference could perhaps be considered vague and general—it would be almost impossible to give details of all the schemes considered and rejected after consultations with the architects and the preparation of comparative estimates—but the simile applied to it would hardly be applicable, because there was nothing to conceal. Given the condition that there had to be the minimum of dust-collecting surfaces, reinforced concrete seemed to be the best material to use, and as had been well established by the rapid expansion in the use of shell concrete for industrial purposes, the scheme adopted proved to be an economical form of reinforced-concrete construction.

On the crucial question of cost, the Authors were in full agreement with various speakers in considering cost the main criterion in judging the success of a design. But Dr Skempton had been right in stressing that it was not the only criterion. One might say that a good solution must be economical, but an economical solution was not necessarily good. That was because any solution, besides fulfilling the main structural function, had a great many secondary qualities, the value of which could not always be assessed in monetary terms. Some, such as ease of maintenance, permanency, additional insulation, adaptability to different uses, etc., would certainly have an economic value, although it might be difficult to assess it; others, to which belong the architectural quality, the psychological effect on the onlooker, the elegance, lightness, or simplicity of the structural solution, would certainly defy estimation in terms of money and even definition, but few engineers would deny that it was some such quality for which they were striving. The engineer was in duty bound to provide it without involving the client in extra cost—or rather, in very much extra cost. The difference was important. Economy was an integral part of that quality, but where it was pursued to the exclusion of other considerations, the effect might be to destroy the quality of the structure for the sake of an insignificant extra cost. Whilst the Authors therefore considered that that undefinable quality should be obtained by sparing no effort in finding the right solution and not by spending money unnecessarily, it would be no service to the client to abandon the search for it altogether in favour of saving the last penny. The point was, of course, that the client and his architect should be the final arbiters in such matters of policy, and the engineers should endeavour to put all the facts of the case before them,

including the extra cost—if any—of schemes which they considered would be a better solution than the cheapest possible.

A question had been asked about the actual cost of the factory. As stated in the article in the *Architectural Review*, the main factory had cost £630,000 (which was £2 8*s*. 1*d*. per square foot) and the boiler-house £50,000—a total of £680,000.

Mr R. S. Jenkins, who also replied, said he had been very interested in the views put forward in the discussion, especially about the shell-concrete domes. He would answer first, so far as he was able, the questions under that heading—in ordinary language and not that of mathematics, because the chief value of the latter was to give one a clearer common-sense picture of how a structure acted than one could obtain without it. Unless analysis achieved that end it was useless and, perhaps, dangerous.

In his opinion, however, the mathematics in the Paper was not as advanced as some of the remarks in the discussion seemed to indicate and he was grateful, therefore, to those speakers who had supplied counterweight by showing that they understood it very well. Nevertheless, it had to be recognized that opinion was fairly sharply divided on the question of mathematics for engineering structures. He wished to say, with deference, that it was not surprising that engineers, who had taken to thinking about shells, should find it convenient to adopt a process of reasoning that looked different from that employed in skeletal, that is, framed, structures. The Brynmawr domes had been standing for some years and they were clearly structures in equilibrium. The part of the Paper dealing with them had attempted to show how data for their design had been obtained by following up the consequences of equilibrium in each and every infinitesimal part. It had been stated why the simple extensional, or membrane, theory had been considered sufficient in that case. It had also been mentioned that there were other cases which demanded, in the Authors' opinion, not only equilibrium but also deformation to be taken into account.

Although Mr Cousins had touched on some very important considerations, his request for an analysis to cover all conditions at the boundary surface would involve the last-mentioned field and would lie well outside the scope of the Paper. Mr Jenkins had indicated in a previous publication (see reference 3, p. 373) a general approach to that field and had also shown that according to the more limited membrane theory, when the boundary was a narrow member curved in plan, the curve of the latter was the string polygon for the outward forces exerted by the shell. It would therefore appear that the only type of curved boundary that could be dealt with by the membrane theory was one that was propped at close intervals. A narrow boundary member, straight in plan, could not in itself take any appreciable outward force and could, as in the case of Brynmawr, also act as an arch from corner to corner. The inward-sloping bow-strings, suggested by Mr Cousins, were certainly an attractive possibility but the simple theory would arrive at tension normal to the boundary in the edge region of the

shell ; whereas, if the additional vertical loads to be carried were relatively large, the top boom of the bow-string would actually be in compression. Mr Jenkins agreed that the incongruities of the membrane theory would, in that case, be so great that it would be unacceptable without modification, and he would advocate an elastic analysis, not because he believed the materials were really elastic and homogeneous, but because it would present a not unreasonable account of stress distribution.

In reply to Professor Baker, the boundary condition assumed for design was that of no outward force, and hence the shell could transmit its loads to the boundary members by shear only. This shearing force increased from zero at the centre of the boundary to maximum at the ends. Although stress trajectories were somewhat deceptive, because they gave little indication whether stresses were small or large, they showed towards the corners equal and opposite principal stresses at 45 degrees, which amounted to shear in the direction of the boundary. Some qualifying remarks on that point were to be found in the Paper.

The Brynmawr domes were made 3½ inches thick because the numerous conduits for the electical wiring to the dome lighting had to be buried, out of sight, in their thickness (see *Fig. 28*). Otherwise they could be 2½ inches as Dr Morice had suggested. The centre of a bow-string on the longer side of a dome deflected by about ⅝ inch when the centering was removed. Dr Morice had also asked for comment upon his model-experiments and calculations on anti-clastic shells. Mr Jenkins said he had become very interested in that type and thought that to cover it by the membrane theory very special and definite boundary conditions had to be provided. He had given his general views in a symposium discussion that had recently been published.[13] It was a case where the idea of a stress function did not seem to serve any useful purpose. He had seen the model in question, and its boundary members did not seem to him capable of acting in a way that would enable the shell to carry its loads mainly by extensional stresses. Perhaps the calculations were saying the same thing, in their own way, by refusing to exhibit convergence.

Mr Measor had questioned whether relaxation was a suitable method for practical design, and Mr Tottenham had shown that, by a slight modification to the shape and loading assumptions of the Brynmawr domes, the governing equation could have had constant coefficients and hence an explicit solution in the form of a Fourier series. That idea was actually the first to be considered for Brynmawr, but it did not seem to the Authors to make any advance in the problem of designing shells of any shape, because the the number of cases which had explicit solutions were very limited indeed. That was why they had become interested in relaxation, which offered a method of solving shapes, otherwise intractable in computation, but obviously structurally sensible. Even classical variational methods usually

[13] "A Symposium on Prestressed Concrete statically indeterminate structures," C.A.C.A. 1951, p. 155.

failed because the limited number of functions that could be found to satisfy the boundary conditions did not converge in the minimizing process. The equation cited by Mr Tottenham provided an excellent example of the power of relaxation, for it could be solved more quickly by one skilled in that art than by the Fourier series. Mr Jenkins added that he personally had developed little skill in the time he had spared for practice in relaxation and looked forward to the use of electronic machines for doing the numerical work. In the meantime if, in the words of Mr Measor, which with he had much sympathy, one did not wish to use " staff who were well qualified to be much better occupied," one could get ordinary relaxation carried out by outside computing services, at a very modest cost. It was not only because they wanted to develop general methods of shell design that the Authors did not proceed with the parabolic translation shell. The more practical considerations that led to the circle of translation were that they wished, in the first embodiment, to keep the principal curvatures as large as possible everywhere, and they thought it might possibly simplfy the centering.

In the last connexion, the Authors had been informed by the contractors that the domes compared favourably in cost per square foot with the cylindrical shells at Brynmawr. Mr Jenkins associated himself with Mr Arup's remark that cost was the main consideration. There seemed to be an impression in some quarters that if one used a little mathematics it increased the cost of the job. His object, whether it succeeded or not, had been to reduce both cost and office labour.

There remained some questions about other parts of the factory. A brief description of the structural action of the cylindrical shells in the northern section would have answered Mr Cousins's question about its expansion joints. Those shells with their slender gable arches were carried by slender columns (see *Fig. 31*) and were all held together against flattening-out by buttress columns at the extreme ends. The general thrust from end to end acted across the expansion joints even though the gables were split in those positions. That type of expansion joint in the shell had been adopted because the latter required increased flexibility over expansion joints in the heavy crane beam and column sub-structure.

Mr Jenkins agreed with Mr Measor's general remarks about the importance of reliable approximations, but he did not know how the extent of an approximation could be measured as a percentage, because nothing in structural engineering was exact. With regard to *Fig. 22*, if the tie, the arch, and the column did not meet at a point, that fact had to be taken into account in the design. The bracket section, h_2, was deep and was assumed to undergo no deformation.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—Sec. I.C.E.

PUBLIC HEALTH ENGINEERING DIVISION MEETING

23 April, 1952

Professor A. J. S. Pippard, Member, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Public Health Paper No. 7

"Bacteria in Relation to Water"

by

Edwin Windle Taylor, M.A., M.D., D.P.H., M.R.C.S., L.R.C.P., Barrister-at-Law

SYNOPSIS

Bacteria multiply rapidly under certain conditions in nature and artificially in special preparations known as media. Water forms an ideal vector for the spread of certain disease-producing intestinal organisms voided in the discharges of man and animals.

Cholera and typhoid are waterborne diseases whilst paratyphoid is more often associated with infected food.

Dysentery and gastro-enteritis, on the other hand, are rarely waterborne and then they are usually due to gross contamination. The spread of leptospiral jaundice and tularaemia is caused by the causative organisms being liberated into the water by rats.

Catarrhal jaundice and poliomyelitis are caused by viruses recoverable from excreta. Whilst jaundice can arise from contaminated water, poliomyelitis has not been shown to do so.

The methods of evaluating pollution evolved by Houston followed the pioneer work of Frankland, Klein, and Boyce, who utilized intestinal organisms as pollution indicators. The prime indicator of excretal pollution is the colon bacillus known in Britain as *Bacterium coli* type I.

Most vegetative organisms are removed by efficient chlorination, for which a water free from ammonia is desirable. This is achieved in sand filters, particularly of the open gravity rapid type, by nitrogen-oxidizing bacteria. Many potable drinking waters contain saprophytic bacteria, but it is desirable that these be reduced to minimal numbers to avoid risk of food spoilage.

It is recommended that water supplies should be tested more frequently, but less exhaustive tests carried out on each sample, and the closest collaboration should be maintained between the various experts dealing with the provision of water for drinking purposes.

Historical Introduction

Water engineering has been carried out by mankind from ancient times and since the days of the earliest settlements, as proved by the Shadoof in the Near East and China, the Pulley and Charsa, and the spiral pump that Archimedes invented more than 2,000 years ago; yet the science of bacteriology is no more than three-quarters of a century old.

It must not, however, be forgotten that Antony van Leeuwenhoek, in the latter half of the 17th century, was the first to see bacteria through his home-made microscope. In a series of letters to the Royal Society, commencing in 1673, he stated how he observed " very little animalcules " in water ; some of these were protozoa and others bacteria. In a letter written in 1683, he describes and illustrates micro-organisms found in material taken from between his own teeth, and the bacteriologists of today are able to identify the bacteria he observed. Leeuwenhoek's researches were recognized by the scientists of his time and he received the highest academic honours. He can be described as one of the world's greatest observers, and Dobell says " He was the first bacteriologist and the first protozoologist, and he created bacteriology and protozoology out of nothing." [1, 2]

Bacteriology as it is known today developed after the introduction of the compound microscope about 1850. It was with this instrument that Louis Pasteur made his great discoveries. He established the relationship between various fermentation reactions and the activity of living micro-organisms ; by his experiments on spontaneous generation he finally disposed of the ancient belief that animals and other living matter could be generated from non-living matter. Later he turned his attention to the study of the causes and prevention of infectious diseases in man and animals.

Man's opinion on the cause of disease has altered from time to time : first it was ascribed to supernatural intervention, then to meteorological disturbances, and for a long time the miasmatic theory held sway that diseases were caused by effluvia emanating from the soil, marshes, decaying substances, or sufferers of the disease. Gradually the idea of communicability of disease arose, and it should be noted that the brilliant researches of John Snow on the mode of spread of cholera and of William Budd on the mode of spread of typhoid fever were completed decades before the causative organisms of these diseases were discovered.

Chief credit for establishing that microbes are the cause of infectious disease must be attributed to Pasteur, but it must be noted that he received considerable support and practical assistance from Joseph Lister, who was able to demonstrate that sepsis of a wound resulted from invasion by bacteria.

Robert Koch, in 1876, was the first to establish in the instance of the anthrax bacillus that a specific microbe can be the causative organism of a specific disease in man and animals. Koch's work on the staining of bacteria and the preparation of cultures of bacteria on solid media changed bacteriology into a science from 1877 onwards. Then followed a great wave of discovery and during the next 20 years the majority of disease-producing bacteria known today were discovered. For example :—

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[1] The references are given on p. 428.

1880 The typhoid bacillus—Carl Eberth.
1882 The tubercle bacillus—Robert Koch.
1883 The cholera vibrio—Robert Koch.
1884 The tetanus bacillus—Arthur Nicolaier.
1885 The colon bacillus (*Bacterium coli commune*)—Theodor Escherich.
1888 The first food-poisoning organism—August Gaertner.
1892 The gas-gangrene bacillus (*Clostridium welchii*)—William H. Welch and G. H. F. Nuttall.
1898 The first dysentery bacillus—Kiyoshi Shiga.

Thus, within a space of 50 years, a new science had been born and one which has proved to be of far-reaching importance in the relief of suffering and reduction in mortality, and has, in fact, produced a great change in the social life and in conditions for the betterment of organized communities.

The Nature of Bacteria

The study of bacteria is not a subject that forms part of our general education and it would not be out of place, therefore, to give a brief outline concerning the nature and behaviour of microbes.

Microbes are commonly known as germs, and scientifically as bacteria. A microbe is a minute unicellular living organism. This simple definition includes three main attributes—bacteria are alive, they each consist of a single cell, and they are small. The characteristics of a living thing are its ability to assimilate substances as food in order to build up its own structure; power of growth; and faculty of reproduction. Although so small, each one is a complete and independent biological entity.

Emphasis must be laid on the extremely small size of bacteria, which can be seen clearly only through the high-powered lenses of the compound microscope. They are amongst the smallest of all living things, so small that a special unit of measurement has to be used, called the micron (μ), which is equal to only one thousandth of a millimetre. A typhoid bacillus measures 3 μ in length, and 0·5 μ in breadth. A mass of germs consisting of some hundreds of millions of individuals will occupy a space no larger than a pin's head. Water appearing to be perfectly clear may yet contain several million bacteria in each drop.

Bacteria are plants but are distinguishable from the majority of other plants by the absence of a definite nucleus; they have no green pigment (chlorophyll), and the cells contain no supporting substance such as cellulose.

The body of a microbe consists of a single cell of living matter and may be one of several shapes—some are spherical, each individual then being termed a coccus; when rod-shaped, the micro-organism is called a bacillus; a comma-shaped organism is spoken of as a vibrio; and a spiral or "corkscrew"-shaped organism is called a spirochaete or spirillum.

Bacterial cells may exist singly or, after division, may remain in characteristic groupings, for example, in chains (streptococcus or streptobacillus), in pairs (diplococcus), packets of four (tetrads), packets of eight (sarcina), or as irregular clusters (staphylococcus). These various shapes and groupings of bacteria always breed true, that is to say, a coccus always produces a coccus and a bacillus always produces a bacillus.

Some bacteria are capable of spontaneous movement in liquids. Their powers of locomotion are caused in some cases by alternate bending and stretching of their bodies, producing a lashing movement which propels them along. More commonly, motility is caused by the presence of fine hair-like projections, called flagella, often as long as the organism itself, which is propelled through the fluid by rhythmical movement of these appendages.

The skin of the cell is usually very thin, but in some species it is thick and of a slimy or mucilaginous texture, in which case it is spoken of as a capsule. Capsulated organisms, by reason of this tough covering, are often difficult to kill. Very frequently such slimy-coated bacteria adhere to one another, forming a gelatinous mass sometimes called a zoogloeal colony.

When conditions for bacterial survival are unfavourable, for example, because of the lack of food, unsuitable temperature, or absence of moisture, certain germs form a tough resistant coat within which the cell is able to hibernate, for years if necessary, until favourable conditions again prevail. These resistant forms of bacteria are called spores and are a means of keeping the organism alive under conditions of adversity. It is important to note that spores are only a resting stage and not a means of reproduction ; only one spore is formed by a cell and only one cell emerges from each spore.

Bacteria usually reproduce by simple division of the cell into two parts. These two newly formed cells undergo similar division in their turn so that, under ideal conditions, one individual becomes many millions in the course of 24 hours. The newly formed cells do not pass through an " infant " stage but are completely and fully formed from the moment of their formation. It is difficult to appreciate how such small entities as bacteria can produce such profound effects, but it is partly owing to this capacity for rapid multiplication because, under the most favourable conditions, one cell is capable of dividing into two every 20 minutes. Although there would only be eight at the end of the first hour there would be as many as $4{,}722 \times 10^{18}$ at the end of 24 hours. If the organism was a bacillus measuring $2\,\mu$ in length, its progeny at the end of 24 hours, if placed end to end, would extend over a distance of one light-year, in terms of astronomical language—which is most certainly an astronomical figure. There is, however, a fallacy in this calculation in that the ideal rate of multiplication continues for only a short time ; although one organism is capable of producing many millions in the course of a few hours, soon the shortage of food and accumulation of waste products begin to take effect, so that the rate of multiplication slows down and finally ceases and,

indeed, the bacterial population, after a stationary period, may die out completely.

Viruses that cause such diseases as foot-and-mouth disease, smallpox, and poliomyelitis are even smaller than bacteria and to measure them an even smaller unit has been introduced; it is a millimicron ($\mu\mu$), which is one twenty-five millionth of an inch. The smallest viruses approach the size of molecules and to give an idea of how much smaller they are than bacteria, if a coccus were enlarged to the size of a tennis ball, a particle of poliomyelitis virus would only be the size of an ink dot on its surface. Size is thus a distinguishing feature between viruses and bacteria; another difference is that viruses only grow in the presence of living cells of the plant or animal they attack. Their true nature is uncertain, but like bacteria they act specifically in transmitting disease; they may be minute living organisms, but some consider them to be non-living.

Bacteria are identified and classified by their appearance under the microscope, by staining with dyes, by cultivation in liquids, and on solids which the bacteriologist calls "media," by their fermentation and other biochemical reactions, and their effect on blood and tissues of animals.

Bacteria exist everywhere—in the air, water, soil, on bodies and clothes, and wherever air, dust, and dirt can penetrate even in minute quantities. The majority of these bacteria are harmless and their action in nature is beneficial. Their function is to keep in circulation the elements necessary for life. Without their activities, which are known as fermentation, putrefaction, and decay, the constituents of living matter would remain stored up, and thus unavailable, in the dead bodies of plants and animals. The following are some examples of the uses of bacteria in nature and how their properties have been adapted in industry :—

(1) Fixation of atmospheric nitrogen in the soil.
(2) Conversion of ammonium salts to nitrates in the soil.
(3) Purification of sewage.
(4) Fermentation of silage.
(5) Conversion of flax fibres into linen threads.
(6) Curing of tobacco.
(7) Manufacture and maturing of beer, wines, vinegar, cheese, lactic acid, and acetic acid.
(8) Formation of glycerol.

In relation to the plant and animal body there are bacteria which have become adapted to their host and which may be called parasitic; these may be either pathogenic, that is, disease-producing, or non-pathogenic, that is, purely parasitic. These forms quite naturally have been studied very extensively in the past in the efforts to alleviate human and animal suffering. Thus bacteriology was first a branch of medicine, but, with increasing knowledge of bacteria as a whole and the vital part they play in nature's economy, bacteriology has become an independent science.

Probably the most primitive types of bacteria are those which can carry on an independent existence in moist places, deriving their essential elements, carbon, nitrogen, oxygen and hydrogen, from inorganic salts, gases, and water. These are known as autotrophic bacteria. Then there are the saprophytic bacteria which live on the organic matter derived from the dead remains of plants and animals. The parasites require a living host on which to thrive and indeed some parasites have acquired the power of invading their host and so are known as pathogenic bacteria. Good examples of the last two divisions are the common colon bacillus (*Bacterium coli*) and the typhoid bacillus (*Salmonella typhi*). Both probably arose from a common ancestor, the former causing no harm in the intestine where it plays a part in the digestive processes, whereas the latter organism, indistinguishable by either microscopical or colonial appearance, penetrates the intestinal wall and invades the blood stream, giving rise to a serious illness followed possibly by death of the individual.

All these bacterial types find their way into water by some means or other and their relationship to water supplies will now be considered, taking them in the reverse order, that is to say, pathogenic first, then truly parasitic, saprophytic, and finally autotrophic types.

Pathogenic Bacteria

Water is a vehicle by which intestinal diseases are spread from one person to another. Discharges of a person suffering from disease reach the drinking water directly or in crude sewage or in sewage effluents, and the specific organisms are dispersed through the water. Of even greater consequence are those persons who have suffered from the disease, but continue to excrete specific organisms during convalescence and even after recovery. They are known as carriers and are the chief means by which epidemics arise. There is also the symptomless carrier who, although quite well in himself or suffering only mildly, nevertheless excretes bacteria and so infects the water.

Cholera

By far the worst of the intestinal diseases is cholera, which has a mortality rate of 50 per cent. The disease is endemic in Lower Bengal and spreads to other parts of the globe from time to time; England was seriously affected in 1831, 1848–49, 1854 and 1865–66. The usual route of infection to Great Britain is from Russia and Germany via our North Sea ports. An outbreak occurred in Egypt as recently as 1947, when there were over 20,000 cases and half of these were fatal.

The last epidemic in England was in 1866, and by then the brilliant deductions of Snow had been recognized, but there was a serious outbreak at Hamburg in 1892, and almost 300 cases occurred in Great Britain as a result. Little has been published about these cholera cases in England in

1893, and a short description of this occurrence may therefore be worth while.[3]

There were 287 cases and 135 deaths from cholera in England in 1893. It was brought into the ports by ships coming from foreign ports and it spread to several districts in England. The most important outbreaks were as follows :—

Grimsby and Cleethorpes.—There was a case on board a ship from Antwerp and an outbreak followed in August and September, resulting in 43 deaths in the area from cholera. There was a simultaneous prevalence of epidemic diarrhoea with a high death rate out of all proportion to other large towns in England during the same period. Cholera was probably present in Grimsby and Cleethorpes some time before the danger was recognized, and the disease secured a footing in those towns for want of adoption of adequate measures of prevention at an early stage. There were also 260 cases and 41 deaths from typhoid fever during this period. The sewage of the district discharged into the sea, but there were cesspools along the lines of the house-drains where sewage could remain and, in addition, the flow of sewage was held back by the tides. It was also possible for the sewage to be washed over the shell-fish beds.

Ashbourne, Derbyshire.—Cases were limited to a single yard behind the Coach and Horses Inn. There were 15 cases of cholera, including 9 deaths, among the 39 persons living there. The inn attracted lodgers of a low class, including itinerants who frequented local fair grounds. Inside the inn and situated against one of its outer walls was a hand-flushing water closet having a broken soil-pipe ; close by was an outside urinal which served, on occasion, as a closet, as also did a gulley outside. In the immediate proximity to these was situated the well which supplied the drinking water. The subsoil was porous and foul to within close vicinity of the well. Dr Klein obtained positive results from discharges and from sections of intestines of the victims and cholera organisms were isolated from the well-water. The outbreak was cut short by stopping the use of the well-water and filling the well with quicklime.

Enteric Fever (*Typhoid and Paratyphoid Fevers*)

Typhoid fever is another serious disease which is typically waterborne, whereas the milder paratyphoid fever is more often spread by infected food, although definite waterborne outbreaks have occurred. It appears that a more massive dose of infective agent is required in the latter case.[4]

Bacteriological technique and sampling have become more efficient in recent years so that it is possible nowadays to isolate pathogenic organisms from polluted water with a fair degree of ease. It is a usual occurrence to find pathogenic organisms in sewage effluents and an instance can be cited of a community in the Home Counties, the sewage effluent from which flows into a stream which eventually flows into a river providing water for London.

A milk-borne epidemic of paratyphoid fever occurred at Epping in 1931, and for the past 21 years paratyphoid B organisms have regularly been isolated from the sewage and in smaller but definite numbers from the effluent. The results are shown in Table 1, which gives a very approximate quantitative picture of the frequency of these organisms in sewage and sewage effluent.

Examination of the effluent was stopped between the years 1941 and 1948, but since that time tests have recommenced and the organisms are readily recovered by modern methods.

A further 22 cases occurred in 1932, and were attributed to cattle entering the stream with consequent infection of the milk from contamination of the cows' udders with the polluted water. There were 2 further cases of paratyphoid fever in 1935, but although the figures given in the Table indicate the high carrier rate of the local community, the general paratyphoid rate of the area is no greater than that for the country as a whole. In other words, there is no abnormal notification of paratyphoid B cases in this locality in spite of the presence of this reservoir of infection.

TABLE 1.—ISOLATION OF PARATYPHOID B BACILLI FROM SEWAGE AND SEWAGE EFFLUENT

Year	*Salmonella paratyphi B* in sewage per millilitre	*Salmonella paratyphi B* in effluent per millilitre
1931	312	57
1932	446	55
1933	219	16
1934	217	7
1935	68	3
1936	102	3
1937	111	2
1938	271	3
1939	318	1
1940	120	1
1941	—	—
1942	—	—
1943	2	—
1944	6	—
1945	17	—
1946	14	—
1947	51	—
1948	39	per litre
1949	53	3
1950	11	6
1951	2	6

Much has already been written about waterborne outbreaks of enteric fever from Worthing in 1893, to Croydon in 1937. The bitter experience of the latter led many water undertakings to examine and correct any shortcomings of their own that might exist, and the Ministry of Health in

1939 published a memorandum on the safeguards to be adopted in the day-to-day administration of water undertakings.[5] This was revised in 1948 and its contents should always be in the minds of those in charge of water supplies.

Since 1937, possibly as a result of greater precaution, no serious direct waterborne outbreak of enteric fever has occurred in the British Isles, although water has been responsible for small groups of sporadic cases and for milk-borne outbreaks. The only waterborne outbreak of any note in this country in recent years was at a Royal Air Force Station in Bedfordshire in 1942, when 22 cases of typhoid fever occurred.[6] The appearance of a high proportion of the cases in one working shed where there were taps and drinking cups available led to a detailed examination of the water supply. The camp received water from the local public mains, and the absence of cases elsewhere made it clear that any pollution must have occurred within the camp. A blocked soil-water drain was located only 3 yards from, and slightly above, the water pipe leading to the shed. The drain was leaking and fluorescein tests proved that contamination might pass to the water pipe within 3 minutes. A booster-pump for fire purposes was run as a routine measure for a few minutes each day and caused rapid fluctuation of the water pressure in the mains ; this might have facilitated entry of the polluted material into the water pipe, the joints of which were found to be " weeping " at this point. No carrier was found, but there had been numerous changes of personnel since the probable date of infection.

One must therefore look overseas to find examples of recent and more serious outbreaks, and the following brief notes illustrate the ever-present risk of typhoid fever if every possible safeguard is not adopted.

(*a*) There was a severe typhoid epidemic in 1939, at Manteno (Illinois) State Hospital for the Insane, which involved 453 cases with 60 deaths.[7] The water supply consisted of 4 partly-lined drilled wells only a few hundred feet apart, in cracked and creviced Niagara limestone covered with a variable layer of glacial drift from 0 to 30 feet. Water discharged into a concrete ground-storage reservoir and was pumped to a distribution system on which pressure was maintained by an elevated tank. Immediately following the installation of the water supply in 1931, the institution officials were notified that samples showed contamination and advice was given to boil the water and install chlorination ; similar warnings were given in 1935 and 1937. There was evidence of blocking of sewers and leaking therefrom in 1939, and there was a significant increase in the chloride content in raw well-water after salt had been added to the sewers at various points. These tests proved that sewage could leak into the creviced limestone and thence into the wells supplying water to the hospital. Further proof was obtained by the isolation of the typhoid bacillus from 6 quarts of raw water collected from one of the wells.

(*b*) In the summer of 1945 a typhoid epidemic occurred in Glion, a small

village of 800 inhabitants in the neighbourhood of Montreux, Switzerland, affecting 101 persons and with 16 deaths.[8] The cause was large-scale contamination of the water supply by leakage from a hotel drain. There was a main drain in the hotel grounds and this had been cracked for a considerable time and had been pierced by tree roots. The contaminated water from this drain leaked into the area of the collecting pipes of the springs supplying the village of Glion. Specific infection was probably derived from a carrier in the hotel, which was being used as a refugee camp in 1945.

(*c*) There was a waterborne epidemic of *Salmonella paratyphi B* in Nueotting, Germany, in 1946.[9] There were 415 cases with a mortality of about 6 per cent and the specific organism was isolated from the town water supply. The epidemic resulted from burst mains (weakened through rust) and contaminated subsoil water and organic matter were allowed to enter them. The organic matter settled on the invert of the mains and absorbed the chlorine used for water treatment, thus rendering it ineffective.

(*d*) There was another epidemic in Germany in 1948, when an outbreak occurred at Goppingen, causing 251 cases of typhoid fever and resulting in 20 deaths.[10] It was found that the filter gallery adjacent to the well and pump had completely lost its covering of sand, so that ground-water entering the gallery was carried rapidly into the central collecting well without any opportunity for purification by filtration. The source of the contamination was probably the heavily manured fields situated less than 200 yards from the filter gallery, and these had been flooded by the heavy rains of January and February. There was also possibility of infection from a leaking cesspool near the pumping station.

Dysentery and Gastro-Enteritis

Outbreaks of waterborne dysentery and waterborne enteritis are rare, but undoubtedly occur. The contamination is usually of a gross character such as occurs when there is a cross-connexion of a pure and a polluted supply or back-siphonage takes place, or where there is accidental influx of sewage into some waterworks structure.

Much has already been published about the Chicago Hotel dysentery outbreak of 1933, so that details will instead be given of a more recent and equally serious outbreak affecting 3,000 persons in the town of Newton, Kansas, in 1944.[11]

Certain repairs were carried out on the water main near a branch supplying a small Mexican village on the outskirts of the town. During this period the water supply was off and consequently there was no positive water pressure in the main. Connecting hydrant and water-closet valves were opened and sewage escaped into the main which was not disinfected after the repairs were completed. The infecting organism was Flexner's bacillus, a member of the dysentery group.

A waterborne epidemic of dysentery due to *Bacterium sonnei* was

recorded from Norway in 1940.[12] The source of infection was found to be water drawn through a suction pipe situated a few feet below the surface of the ground in a district which had recently become popular for winter sports.

In Germany in 1947, an epidemic of dysentery was traced to an infected well.[13] The organism was isolated from the water by membrane filtration technique and was the same species as that isolated from the excreta of the infected persons.

Bacterial food-poisoning or gastro-enteritis is caused by organisms of the Salmonella group which cause disease or may live harmlessly in the intestines of animals and are consequently found in their excreta. These organisms are distinguished from the paratyphoid bacteria essentially by their serological reactions; new serotypes are constantly being isolated and it appears that almost every species so far described is capable of infecting human beings, the most common vehicle of infection being food, but as will be seen shortly, water is not exempt. Whilst chronic human carriers are very uncommon it would appear that a temporary carrier state of a few months' duration may occur in man. The organisms are frequently isolated from sewage effluents and polluted river-water and a mixed outbreak involving cases of dysentery and gastro-enteritis has recently been reported.[14]

A factory in the Midlands had two supplies of water—the town's main supply and an unpurified river supply used in manufacturing processes. The dirtiness of the river supply eventually rendered it unsuitable for use and a connecting pipe was placed between the river-supply pipe and the town-supply pipe to provide purified water for factory purposes. Unfortunately, the isolating valve fitted to prevent any communication between the two supplies had been left open and since the river-water was at a higher pressure it was driven into the water company's mains. Illness appeared among the factory workers 3 days later and ultimately a total of 160 employees at the factory and 28 persons in neighbouring houses became ill with diarrhoea and vomiting. Dysentery and food-poisoning bacteria were isolated from stools of the sufferers and the dysentery organism was later isolated from the river-water.

Eliassen and Cummings [15] have analysed the waterborne outbreaks in the United States of America between 1938 and 1945, and in a list of over 300 outbreaks involving more than 100,000 persons, 99 were caused by typhoid fever, 35 were dysentery, and 198 (more than one-half) were gastro-enteritis, which was responsible for most of the cases. Contamination of the distribution system through cross-connexions and back-siphonage was responsible for two-fifths of the cases.

Weil's Disease

The varieties of spirochaetes found in water are, with one exception, harmless to man; the one in question is that causing Weil's disease or

leptospiral jaundice. Rats harbour the organisms and contaminate the water. The organism is capable of penetrating the skin so that infection is possible otherwise than by drinking water, for example, through wading —it appears from time to time among sewer workers,[16] or by bathing. Seven cases of Weil's disease occurred near Hamburg in 1949, all patients having bathed in a stagnant pool the surrounding ground of which was infested by rats.[17]

Tularaemia

In recent years, several outbreaks of tularaemia have been proved to be waterborne. The condition is characterized by general fever for 2 or 3 weeks, glandular involvement, skin eruptions, and prolonged convalescence. This is another disease the organism of which is primarily found in rodents and, in particular, in rats which may be the cause of contamination of water. An outbreak occurred in Russia in 1946, where river-water was used for the local drinking-water supply after the artesian well had been destroyed during the war.[18] In 1949, 12 cases were reported in Montana and the specific organism *Brucella tularensis* was isolated from streams in the area and, in addition, the domestic water supply at the residence of one case was twice shown to be contaminated with the organism.[19]

Virus Infections

Two diseases caused by viruses must now be considered—catarrhal jaundice and poliomyelitis. The viruses are present in the excreta of cases and otherwise apparently normal carriers, but little is known as to their mode of spread. It may well be that the whole concept of the epidemiology of virus diseases differs from that of the bacterial diseases and until this has been explored the present incidence, distribution, and mode of spread will remain a mystery.

Catarrhal jaundice is also known as epidemic infective hepatitis. It was prevalent among the armed forces during the 1939–45 War, and polluted water has been held responsible in one outbreak at least. More than 350 individuals suffered from the disease in a camp where one of the wells was proved to be contaminated from a nearby cesspool.[20] The infective agent was proved to be present in faecal specimens from patients, and in the well-water by production of typical jaundice or evidence of hepatic disfunction in volunteers. It was therefore concluded that the cases were probably caused by oral infection from drinking well-water contaminated by the sewage from cesspools which contained the virus of infective hepatitis, which had apparently survived for at least 6 weeks.

Two other good examples of probable waterborne outbreaks deduced solely on epidemiological grounds have been reported from North Sweden (1944–45),[21] and Italy (1945).[22] In both these cases there was gross faecal infection of the drinking-water supply.

The association of poliomyelitis and drinking water is often mentioned, and it seems opportune to record the extent of any relationship in the light of present knowledge.

It was for a long time believed that the disease belonged to the respiratory group and infection was spread by droplet infection, but since 1940 it has been demonstrated that the virus is discharged in the excrement of infected persons and is capable of surviving and retaining its virulence for long periods in sewage and in water. It has even been detected in sewage effluent, but not beyond this point in the external environment. Theoretically, therefore, it is possible for the poliomyelitis virus to be transmitted in polluted water supplies. The virus, however, has never been isolated unequivocally from naturally infected water supplies, but it has been shown to persist for 180 days and more in artificially infected river-water stored in the dark and at 4° C.[23]

Authoritative statements on poliomyelitis and public water supplies have been made by Maxcy[24] and Mackenzie.[25] Epidemiological studies show that there is no record of a single outbreak of this disease which could be conclusively traced to the simultaneous exposure of a group of people to a common water supply, nor have a series of cases been linked together with water as the sole aetiological factor. Again, there is no evidence to show that the virus of poliomyelitis has ever been distributed in a public water supply in a virulent state or in a sufficient quantity to cause infection nor that it is capable of surviving the purification processes commonly applied to water meant for human consumption.

The epidemiological picture is confused by difficulties of isolation and identification of the organism, the numbers of healthy carriers during an epidemic, and the existence of a high degree of immunity. Most of the experimental study of the virus of poliomyelitis has been confined to laboratories specially equipped for the purpose, and has for the most part been carried out in the United States of America. A very important investigation carried out in 1949[26] was the estimation of the concentration of chlorine and chloramine required to inactivate the virus. Although these were carried out on a laboratory scale they were designed, so far as was possible, to simulate conditions prevailing in waterworks practice. The results, expressed in a few words, indicate that the virus is inactivated in 10 minutes by a residual of free chlorine of 0·05 part per million and in less than 2 hours by a residual of chloramine of 0·75 part per million. If these findings are not contradicted, the final treatment of domestic water supplies to give a residual of free chlorine should provide an adequate barrier against the transmission of the virus. Nevertheless, the question of adequate safeguard must be kept constantly under review as contributions are added to the present knowledge and conception of the spread of virus disease.

Early Water Bacteriology

Until the appearance of bacteriology, reliance had to be placed on the chemical analysis of water for the assessment of purity and, from the hygienic point of view, the results were disappointing. Water would fulfil the strictest tests for chemical purity and yet contain sufficient contamination to cause widespread typhoid fever. From 1882 onwards, the new science of bacteriology was applied to water in Great Britain by Percy Frankland and Emanuel Klein, who used Robert Koch's gelatine-plate method for the quantitative estimation of bacteria in water.

Percy Frankland, in 1885, commenced the first routine bacteriological examination of London's water supply, and in 1891 he made a most important pronouncement when he stated that organisms characteristic of sewage must be looked for to provide evidence of dangerous pollution.

Robert Boyce, in 1898, commenced systematic examinations for bacteria in the Liverpool waters, and his method consisted of the quantitative estimation of the total number of bacteria present together with a qualitative examination for two commonly occurring intestinal organisms, namely, " Bacillus coli " and " Bacillus enteritidis sporogenes."

It was left to Alexander Houston to show that it was not sufficient to demonstrate the mere presence of intestinal organisms in water, but their numbers should be estimated in order to gain some idea of the degree of pollution. Houston showed that the bacteriological method was from ten to a hundred times more delicate than the chemical method, and from the turn of the century onwards he devoted the whole of his life to water supplies and developed the bacteriological method of water analysis which, apart from minor modifications, is still in use today.

Perhaps one other name should be mentioned among these pioneers and that is Alfred MacConkey who, in 1900, introduced the medium which goes by his name for the differential isolation of intestinal organisms.

It is of the first importance to appreciate why bacteriologists do not examine a water as a routine measure for the presence of pathogenic intestinal organisms, as a method of assessing the hygienic quality of a water. In spite of the introduction of efficient specific media for the isolation of intestinal pathogenic organisms and improved sampling technique, the water bacteriologist is unable to assess the purity of a supply by demonstrating the absence of pathogenic organisms because of the technical difficulties involved as a result of the sparse distribution of these organisms, if present, and the relatively long period of time required for their isolation. In addition, there is always the possibility that, if pathogenic organisms are found, the information may arrive too late because the water will already have been drunk. As was recognized in the closing years of the 19th century by Frankland, a much wider margin of safety must be applied for assessing and establishing the purity of drinking water, in that it must be ensured that organisms normally present in the human and animal intestine,

and therefore indicative of excretal pollution, are themselves absent from the water. It is through this wise dictum, made and followed so many years ago, that drinking-water supplies are maintained at such a high degree of purity today.

Parasitic Bacteria

The routine bacteriological assessment of the purity of a water is determined by the enumeration of harmless parasitic bacteria which are normal and constant inhabitants of the human and animal intestine. These faecal bacteria may therefore be looked upon as indicators of pollution and as danger signals because, if pathogenic bacteria are associated with them in the sewage or other source of contamination, the path is open for them to appear in the water. If the sewage-indicating bacteria are not present then it can be concluded with confidence that the water is safe.

There are three main groups of bacteria which are utilized in this manner as indicators of excretal pollution :—

1. The genus Bacterium or the Coli-aerogenes group for which, in accordance with well established usage, the term " coliform " is often employed. These bacteria are closely allied to enteric, dysentery, and food-poisoning organisms, but in ordinary circumstances they do not give rise to disease. One member, namely, *Bacterium coli* type I, is by far the most frequent coliform type present in the human and animal intestine ; it is found in enormous numbers (one hundred million to one thousand million per gramme), and no adequate evidence has been advanced to prove the existence of *Bacterium coli* type I in sites entirely removed from any possibility of faecal pollution. It is maintained, therefore, that *Bacterium coli* type I is found only in waters which are liable to direct or indirect contamination by material originating in the human or animal intestine. In addition, since *Bacterium coli* type I does not flourish in the external environment, but on the other hand slowly succumbs, its presence in sufficient numbers demonstrates that pollution is continued or fairly recent. Waters from sources subject to no immediate serious pollution are generally free from *Bacterium coli* type I. Unfortunately, no satisfactory method is at present available to distinguish between *Bacterium coli* type I strains of human and of animal origin, but it must be appreciated that the real danger towards supplies is not the presence of *Bacterium coli* type I themselves, but the possible association of other disease-producing organisms. It acts, therefore, as an indicator or danger signal, to give warning that pathogenic bacteria may also be present. In this respect human excrement is by far the most dangerous, but since animals can be carriers of human infective agents it is rightly assumed that all cases of excretal pollution of water supplies are potentially dangerous to its consumers.

Other members of the coli-aerogenes group—intermediates, aerogenes,

and cloacae (I.A.C.) types—are found to a much lesser extent in the intestines, but their ecology has not yet been finally established. The I.A.C. group tend to live longer in the external environment than *Bacterium coli* type I and, in conditions of raised temperature, may even multiply. Intermediates are more often associated with the soil, and aerogenes-cloacae types with vegetation.

The fact that *Bacterium coli* type I is constantly found in the intestine was shown both by Mackenzie and Hilton Sergeant,[27] and by Bardsley,[28] who found *Bacterium coli* type I in every specimen of faeces examined by liquid enrichment methods. The relative preponderance of *Bacterium coli* type I compared with other coli-aerogenes species is also shown by the results of direct plating of faecal material and the examination of the resulting individual colonies. Taylor [29] examined 780 samples, and *Bacterium coli* type I was present in 91·7 per cent, whilst I.A.C. organisms were found in only 13·4 per cent. It cannot be said from these results that I.A.C. organisms are any more than organisms of passage or " fellow travellers " in the intestine, being introduced chiefly in the food.

2. The Faecal Streptococcus or Enterococcus, as its name implies, is a round organism and exists in chains ; it is closely related in morphology and staining reactions to the germs causing tonsilitis, scarlet fever, puerperal fever, and blood poisoning. Its natural habitat, like that of *Bacterium coli* type I, is the intestine, but they are usually fewer in number. They are not found in pure waters nor in other sites out of contact with human and animal life, nor do they multiply outside the animal body, except in milk and similar foodstuffs.

3. The gas gangrene bacillus or *Clostridium welchii* is an organism capable of thriving in the absence of oxygen, and of existing in a resistant phase called a spore, so that it can outlive many other bacteria and usually resists chlorination. It also, is a normal inhabitant of the intestine, but fewer in number than *Bacterium coli* type I. Soil and mud, wherever exposed to sewage or manurial contamination, even considerably remote, are infected with clostridial spores.

Significance of the Bacterial Sewage Indicators

There is general agreement that *Bacterium coli* type I is the prime indicator of excretal pollution, and if found in water it indicates recent contamination and the necessity for applying purification measures, in order to render the water safe to drink. It has already been stated that *Bacterium coli* type I far outnumbers I.A.C. organisms in the intestine so that the presence of I.A.C. organisms, at the worst, indicates remote excretal pollution.

When using bacteria as indicators of the efficiency of various purification procedures the *Bacterium coli* content is a sufficient guide as to bacterial purity in the raw water and at each stage of purification until the final stage is reached, namely, final chemical treatment when, if this

be efficient, all coli-aerogenes types should be removed. Thus, for example, there is such little difference between the resistance of *Bacterium coli* type I and I.A.C. organisms to chlorination that the recovery of any indicates that the chlorination procedure is at fault. In fact, efficient chlorination should remove all ordinary vegetative bacteria.

The difficult problem is to assess the significance of coliform types in untreated water when typical *Bacterium coli* type I is absent. It is considered that a little more space is worth devoting to this topic, for it is of a contentious nature, but without detracting in any way from the singular importance of *Bacterium coli* type I.

Gray [30] found the aerogenes-coli ratio relatively low in recently fouled water but that the ratio is rapidly reversed by storage. He found that the proportion of aerogenes to coli in the Liverpool water-supply is relatively high and increases with storage. He concluded that a preponderance of aerogenes over coli in a water sample is indicative of contact with soil not contaminated with fresh faecal matter or of long-past faecal contamination and, for practical purposes, of freedom of the water from pathogenic organisms. Mackenzie and Hilton Sergeant [27] have expressed the opinion that the presence of aerogenes in the absence of *Bacterium coli* type I may be accepted as evidence that any pollution which might have existed is so remote as to be harmless.

In support of the possible faecal significance of I.A.C. organisms, it is sometimes found that a low I.A.C. count may be followed in subsequent samples by the appearance of *Bacterium coli* type I, so that even low I.A.C. indices may be precursors of more serious pollution. The following are two actual examples which illustrate the point :—

(*a*) Table 2 indicates the results of bacteriological analysis of water from a service reservoir : after a few samples having low I.A.C. counts, samples containing *Bacterium coli* type I soon followed.

TABLE 2.—RESULTS OF WEEKLY BACTERIOLOGICAL SAMPLING OF A SERVICE RESERVOIR

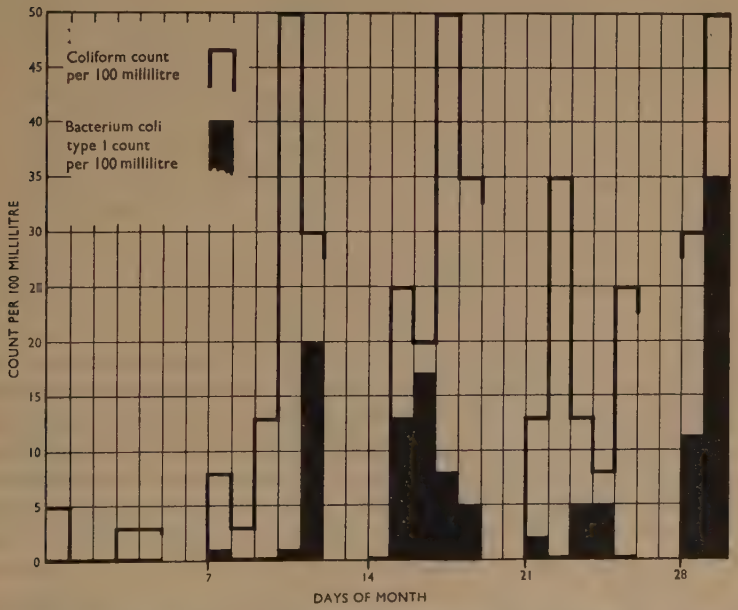
Weeks	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Coliform count per 100 millilitre	0	0	0	0	0	0	0	2	0	2	3	1	16	1	6	25	25
Bacterium coli count per 100 millilitre . . .	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	25

(*b*) The second illustration is a borehole in the chalk, the water from which is known to be polluted, and an increase of contamination can be expected within 24 hours after rainfall. The area surrounding the site is on the bare chalk, is becoming increasingly urbanized, and contains a number of cesspools and ponds.

Fig. 1 illustrates the coliform and *Bacterium coli* counts during the period of 1 month, when it will be seen that *Bacterium coli* soon appear after a few samples have shown relatively small numbers of I.A.C. organisms.

It will be seen that, if the well had been examined on only 1 day each week, for example, days 1, 8, 15, and 22, the coliform/*Bacterium coli* counts expressed as a ratio would have been 5 : 0, 8 : 1, zero, and 13 : 2 respectively. Whereas by sampling more frequently the overall bacteriological picture of the well is seen to be much worse and furnishes a good example of how a low coliform count composed of I.A.C. organisms may be the precursors of typical *Bacterium coli*.

Fig. 1



Coliform and Bacterium coli Counts in Water Samples from a Borehole in the Chalk

On the other hand, there is another type of case where the coliform counts are markedly high, with I.A.C. types predominating and, if *Bacterium coli* occasionally appear, their numbers are very few. Such an example is shown by the results from a shallow borehole and set out in Table 3. The presence of these organisms is probably the result of multiplication of I.A.C. organisms in soil and on vegetation. Although some of these coliform counts are high and indicate the ingress of soil organisms, the bacteriological picture is less serious from the epidemiological point

of view than in examples (*a*) and (*b*) because of the relative absence of typical *Bacterium coli.*

Of course, every source showing I.A.C. organisms should be followed up and their origin traced, and even when no *Bacterium coli* are found it is often possible by relatively simple measures, such as attention to coping, lining, covering, and surroundings of a well, to reduce the coliform count to a level of between 10 and 25 per 100 millilitres.

TABLE 3.—RESULTS OF WEEKLY BACTERIOLOGICAL SAMPLING OF A SERVICE RESERVOIR

Weeks	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Coliform count per 100 millilitre	50	160	8	5	25	250	50	20	14	35	8	300	0	5
Bacterium coli count per 100 millilitre	0	8	0	0	0	0	0	0	0	0	0	0	0	0

Weeks	15	16	17	18	19	20	21	22	23	24	25	26
							*				*	*
Coliform count per 100 millilitre	11	13	13	50	160	90	–	25	25	900	–	–
Bacterium coli count per 100 millilitre	0	0	0	0	0	0	–	0	0	0	–	–

* No samples examined.

There are often cases, however, where the coliform count in a water supply is found to be derived from an innocuous source and has no epidemiological significance. As an illustration, water from a dug well persistently gave high coliform counts of the order of several hundreds per 100 millilitres, and the organisms responsible for the presumptive test also gave positive fermentation of lactose at 44° C., which in Great Britain is almost invariably indicative of *Bacterium coli* type I. However, on plating these cultures and finally identifying the colonies they were found to be a pure culture of an aerogenes-like organism capable of growing at 44° C. Subsequent investigation of the well and headings showed that this organism was actively growing on the pump-packing (jute), and on decomposing wood-blocks fixed in the chalk, whereas it was not found in the water issuing into the adits from the boreholes and, in fact, the water was of excellent bacteriological quality. The organism in question is uncommon in Great Britain, being confined chiefly to jute and hemp, but it is frequently isolated from faecal matter in India and it would appear that it is imported into Great Britain in the raw jute. The infected jute was used to pack the well-pump and the organisms gained access to the well-water and had developed on woodwork in the well.

Thus the significance of coliform organisms other than *Bacterium coli*

type I in water supplies can in no way be related to the absolute number present. When a few appear for the first time in water from which they have previously been absent, they may be the forerunners of dangerous pollution, whereas persistently high counts may, as in the case just mentioned, be derived from some innocuous source and have no epidemiological significance whatever.

Turning now to the other groups of bacterial indicators of excremental pollution, it is noted that the faecal streptococcus is not looked for in routine water analysis as a general rule for, although it is indicative of excremental pollution, it is usually present in fewer numbers than *Bacterium coli* type I and, if the latter is isolated from the sample, it is unnecessary to look for streptococci. Thus the streptococcus test is entirely subordinate to the *Bacterium coli* test, and a negative result in the former case does not lessen the significance of an accompanying positive *Bacterium coli* finding. Its chief value lies in assessing the quality of a water when the results of the coliform test are inconclusive ; for example, if the coliform types isolated from a water give irregular reactions or are not conclusively faecal in origin, then the examination for, and the isolation of, faecal streptococci will be evidence that the water is dangerously polluted.[31]

The importance of the *Clostridium welchii* test in water analysis is solely due to its ability to resist an adverse environment by means of its spore or resting stage, so that it may be detected in a water long after other indicators of pollution have died out; it therefore serves as an indicator of intermittent or remote pollution, particularly in connexion with well supplies. Occasional isolation of a coliform organism may not be significant, but the demonstration of *Clostridium welchii* will show that the water is subject to contamination, and that safeguards are necessary to protect the consumer. Chlorine in doses normally used in waterworks practice does not destroy the spores of *Clostridium welchii*, hence the test based on the isolation of this organism is of no use in assessing the efficiency of chlorination.

Saprophytic Bacteria

The bacteria found in water are those naturally occurring and those acquired during its passage through the air as rain, or as it flows over the ground from soil and other surfaces, or from more heavily polluted sources such as sewage, sewage effluents, etc. Thus, one can speak of a clean water and a dirty water from the point of view of its total bacterial population just as one does in the case of milk. The enumeration of the total bacteria in a sample is known as the Colony Count, and is made by adding a known volume of water to a nutrient medium which is capable of solidifying at the desired temperature of incubation. Under these conditions of adequate food material, those organisms embedded in the medium that are capable of thriving at the particular temperature of incubation will

multiply rapidly, and will eventually develop into a colony visible to the naked eye. The colonies can then be counted, and it is assumed that each colony represents a single organism in the original water sample, an assumption which is reasonably true in the case of water. Thus the colony count is an estimation of the number of bacteria present in a water which will grow on a particular medium after a specified period of incubation, and at a specified temperature. Some organisms will only grow on certain media or at certain temperatures, and vary in their rate of growth, so that it will be seen that colony counts can be made in a variety of ways, each one yielding a different result. It is now standard practice in the bacteriological examination of water supplies in Great Britain to use nutrient agar as the culture medium and incubate one series of tests at 37° C., that is, at blood heat, and another series at 22° C. The 37° C. count gives an estimate of the organisms capable of growing at blood heat and, although most of these are saprophytic organisms derived from soil and sewage, the count will also include parasitic types. The 22° C. count, on the other hand, enumerates mainly harmless saprophytic organisms many of whose natural habitat is the water itself. It should be realized that neither the 37° C. nor the 22° C. counts purport to represent the total number of bacteria present, but merely indicate the number of viable bacteria in the water that are capable of growing on nutrient agar within the time of incubation and at that particular temperature.

The colony counts are of chief value when carried out regularly on the same supply when any increase may be considered significant and to justify further investigation. It is impossible to lay down standards of quality from the point of view of total number of bacteria present, because of the variation in the numbers present in different sources of supply and the fact that a water may be of satisfactory hygienic quality and yet contain a number of saprophytic bacteria. For example, raw chalk well-water may regularly be free from organisms capable of growing at 37° C. in 10 millilitre of water, but equally safe is a river-derived filtered water which has been chlorinated and yet may still contain many aerobic spore-bearing organisms that are not destroyed by the final chlorination process.

It is emphasized, however, that every effort should be made, whatever the source, to reduce the total bacterial population of a finished water so far as is possible, because a bacterially clean water is more highly desirable from the hygienic point of view than a dirty water. Beyond the bounds of a safe water the water supply should contain few bacteria, in order to reduce the possibility of food spoilage, for example, in the dairy industry and when articles of food have to stand in contact with water. If the latter is heavily infected, the organisms are likely to develop on the food and make it unfit to eat.

Water Analysis

The routine bacteriological analysis of water relies on the examination of the indicators of faecal pollution and the colony counts at 37° C. and 22° C. In the past it was the fashion to sample a supply at relatively infrequent intervals and to carry out a full investigation for all the above-mentioned organisms. The tendency today is to collect samples more frequently but to carry out only the more important tests upon them; in other words, "fewer tests but more often." Epidemics that have been traced to water have often resulted from accidental causes such as the sudden breakdown in purification, or sudden grave pollution from flooding, or from broken drains or sewers. The trouble has not been detected immediately and it is only by more frequent and regular sampling that such tragedies can be detected early and so averted.

A complete analysis is reserved for investigation of new supplies and is only part of the main investigation which includes history of the source, topographical inspection, and other analytical examinations. The *Bacterium coli* test alone is sufficient for checking the efficiency of each stage of the purification process in a waterworks, but these tests should be made frequently in order to check and control any falling off in efficiency of the plant. Samples of finished water passing into supply or of water in the distribution system should be subjected to the coliform/*Bacterium coli* counts and the 37° C. count. Water leaving the works should be examined daily or at least five times per week, and if more than one purification point exists in the particular system then each particular point should be so examined.

Autotrophic Bacteria

The fourth group of bacteria are the autotrophic bacteria which, as already explained, include that great number of organisms in nature which obtain their food and energy requirements from gases and simple salts in water. Of these only two kinds in relation to water have been dealt with in the Paper.

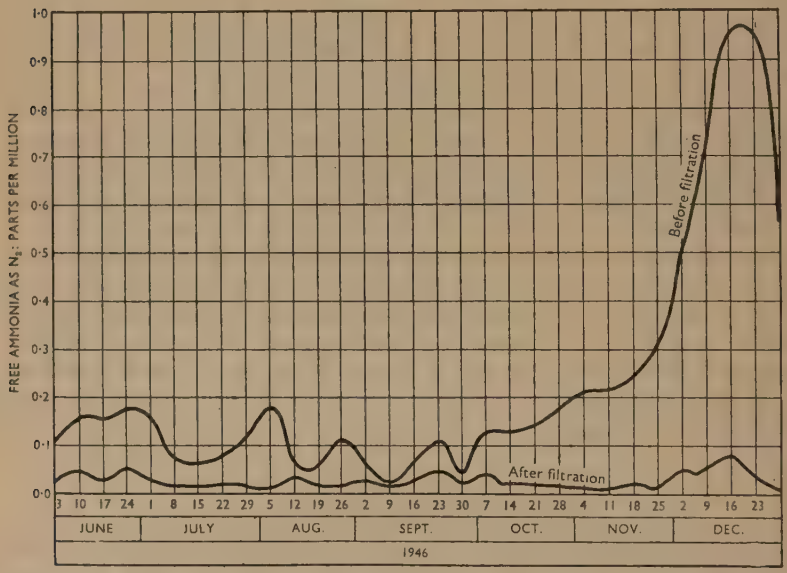
Nitrogen-Oxidizing Bacteria

It has long been known that there is microbiological oxidation of free ammonia to nitrites and nitrates in the soil, in percolating filters of sewage works and in sand filters of waterworks, but it is only in recent years that the extent of this process has been appreciated as it occurs in rapid sand filters operated in the manner employed by the Metropolitan Water Board. The complete removal of ammonia from water is now considered an essential requirement in water purification in order to achieve efficient chlorination, free chlorine being so much more powerful and rapid in action than the products of chlorine with ammonia.

The sand grains in a rapid filter are between 0·5 millimetre and 1·0 millimetre in diameter, and when a filter has matured the grains become coated with a soft precipitate of calcium carbonate and a gelatinous film made up of bacteria prominent among which are the nitrogen-oxidizers, of which there are two groups—Nitrosomonas and Nitrobacter, named by Winogradsky, who first isolated them in 1890–91.

Water passes through these open gravity filters without coagulant at rates ranging between 100 and 200 gallons per square foot per hour and if the slower rate is considered it is found that a drop of water takes only

Fig. 2



Reduction in Free and Saline Ammonia during Passage of Water through Rapid Filters

15 minutes to pass through a layer of sand and gravel 4 feet in depth. During this transit almost all the ammonia in the water is oxidized by bacterial action, the amount remaining being of the order of 0·01 part per million (N_2), an amount that does not interfere with chlorination. Rapid filters are able to remove 0·5 part per million within this time and in this manner with regularity, and, as will be seen later, are capable of removing even more should the occasion arise.

Fig. 2 shows the amount of ammonia expressed as nitrogen in water passing on to the filters and the amount remaining after filtration ; the results are taken from the records of the performance of the rapid filtration plant at the Stoke Newington Works of the Metropolitan Water Board during the last 7 months of 1946.

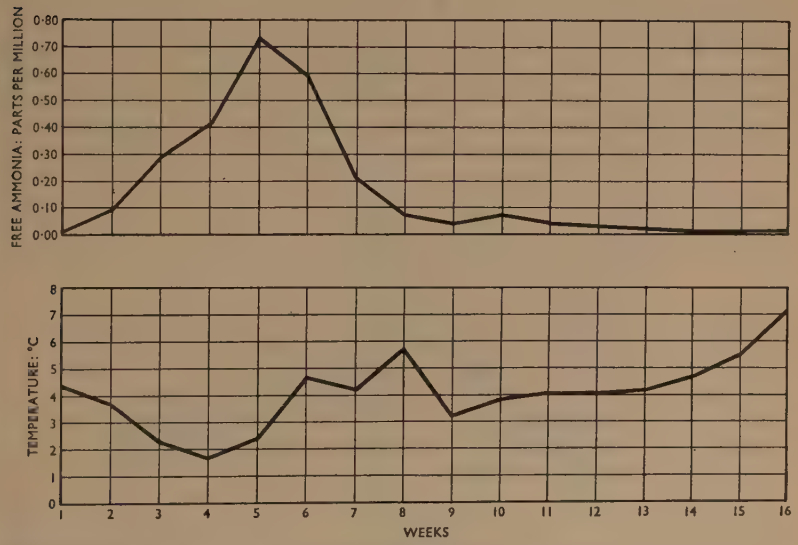
Two factors react unfavourably towards ammonia-oxidation in sand filters.

The first is temperature ; it is found that below 4° C. the microbiological action appears to be temporarily suppressed only to start up afresh with extreme vigour when warmer conditions prevail.

Figs 3 illustrate the effect of a fall in temperature on the ammonia content of the primary filtrate, and it will be seen that there is a tendency to rise when the temperature falls below 4° C.

Fig. 4, which is a continuation of *Fig. 2* on a smaller scale, shows

Figs 3

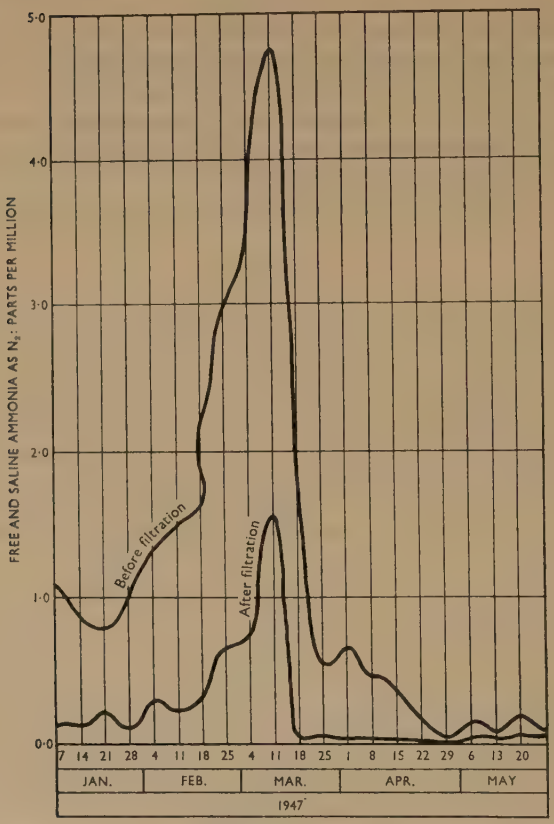


THE EFFECT OF A FALL IN TEMPERATURE ON THE AMMONIA CONTENT IN A PRIMARY-SAND-FILTERED WATER

the ammonia content in the unfiltered and filtered water in the Stoke Newington rapid filters during the first 5 months of 1947, the early months of which are memorable for the prolonged period of arctic conditions followed by extensive floods. It illustrates the partial breakdown in the ammonia oxidation process as the temperature fell, but it will be noted that on the 11th March, when the water temperature was just less than 1° C., as much as 3 parts per million of ammonia were being oxidized and the power of this process is most manifest when it is appreciated that all this occurred in the course of 15 minutes.

The second unfavourable factor is the presence of residual chlorine in the water flowing on to the filters ; any amount exceeding 0·2 part per million appears to have an adverse effect on their efficiency.

Fig. 4



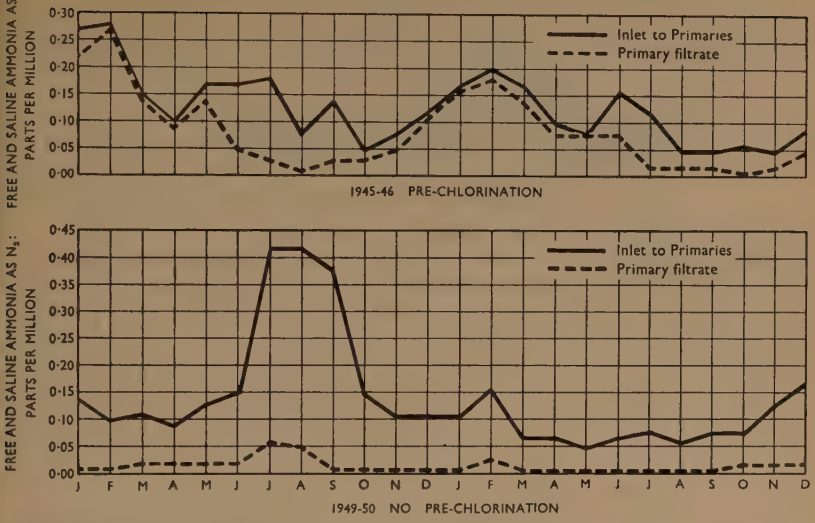
Reduction of Free and Saline Ammonia during Passage of Water through Rapid Sand Filters

Table 4 and *Figs 5* illustrate this point by showing the amount of ammonia removed from Kempton Park primary filters in the years 1945 and 1946, when chlorination of the pre-filtered water was being carried out as a routine practice, and the years 1949 and 1950 when pre-chlorination had been suspended. Expressed in figures, when chlorine was present in the water being passed on to the filters, only 34 per cent of ammonia was being oxidized, but, after the cessation of the chlorination process, 88 per cent of the ammonia was oxidized.

For their continued efficiency, nitrogen-oxidizing bacteria need oxygen and a well-oxygenated water, and it may well be that the regular daily practice of scouring the sand with compressed air provides a means of maintaining the supply of this vital oxygen.

One final point concerning these bacteria is that they require a plentiful

Figs 5



Effect of Prefiltration Chlorination on Ammonia Oxidation by Kempton Park Primary Filters

Table 4.—free ammonia (parts per million)

(*a*) *Chlorination prior to filtration*

	1945		1946	
	Ammonia (p.p.m. N_2) Inlet to Primary filters	Ammonia (p.p.m. N_2) Primary filtrate	Ammonia (p.p.m. N_2) Inlet to Primary filters	Ammonia (p.p.m. N_2) Primary filtrate
Jan. .	0·27	0·22	0·17	0·16
Feb. .	0·28	0·27	0·20	0·18
Mar. .	0·15	0·14	0·17	0·14
Apr. .	0·10	0·09	0·10	0·08
May .	0·17	0·14	0·08	0·08
June .	0·17	0·05	0·16	0·08
July .	0·18	0·03	0·12	0·02
Aug. .	0·08	0·01	0·05	0·02
Sept. .	0·14	0·03	0·05	0·02
Oct. .	0·05	0·03	0·06	0·01
Nov. .	0·08	0·05	0·05	0·02
Dec. .	0·12	0·11	0·09	0·05

TABLE 4 (CONTINUED).—FREE AMMONIA (PARTS PER MILLION)

(b) No pre-chlorination

	1949		1950	
	Ammonia (p.p.m. N_2) Inlet to Primary filters	Ammonia (p.p.m. N_2) Primary filtrate	Ammonia (p.p.m. N_2) Inlet to primary filters	Ammonia (p.p.m. N_2) Primary filtrate
Jan. .	0·14	0·01	0·11	0·01
Feb. .	0·10	0·01	0·16	0·03
Mar. .	0·11	0·02	0·07	0·01
Apr. .	0·09	0·02	0·07	0·01
May .	0·13	0·02	0·05	0·01
June .	0·15	0·02	0·07	0·01
July .	0·42	0·06	0·08	0·01
Aug. .	0·42	0·05	0·06	0·01
Sept. .	0·38	0·01	0·08	0·01
Oct. .	0·15	0·01	0·08	0·02
Nov. .	0·11	0·01	0·13	0·02
Dec. .	0·11	0·01	0·17	0·02

supply of ammonia to keep up their population, and it is found that during the summer months, when the ammonia content of the unfiltered water is low, their numbers fall. In addition, there is the fact that chlorination for algal control or other purposes may be taking place in reservoirs upstream of the filters, so that fresh supplies of nitrogen-oxidizing bacteria are killed before they reach the filter and so prevent any natural inoculation of the sand. Under these conditions it is found that in the early part of the winter season the ammonia oxidation process is less efficient, and it is therefore advisable to ensure a fresh inoculation of the filters by exposing them to untreated water in preparation for the time when the ammonia content rises in the water.

Filamentous Bacteria

Filamentous bacteria may be present in pure or polluted waters. They consist of the filamentous sulphur bacteria and the sheathed bacteria, which have been popularly termed the "iron" bacteria and the "sewage" bacteria. The variability in the appearance of the sheathed organisms has made identification difficult and caused some confusion in the nomen-

clature. Pringsheim,[32] as a result of his experimental work, concludes that there are only two genera, Sphaerotilus and Crenothrix, and they are differentiated by their method of reproduction. Only a small amount of organic matter, if any, is required for the development of these organisms and the covering sheath is thin under such conditions, but in the presence of ferrous or manganous compounds there is a considerable increase in the amount of cell material formed, so that the sheaths become thickened, being composed of iron or manganese hydroxides.

The organisms are harmless from the hygienic point of view, but their proliferation gives rise to slimy grey or reddish deposits in tanks and pipes ; taps, valves, and meters may be blocked and the appearance of tufts of this material in water from domestic taps will give rise to complaints from consumers. A recent outbreak of filamentous growth is described by Mackenzie [33] in water from a chalk well. It was known that water in a disused gravel-pit 50 yards away was connected to the well by fissures in the chalk. Complaints were received that bits of white material like cotton wool were issuing from domestic taps and that meter boxes were becoming clogged with the material. Microscopical examination revealed a specimen of Crenothrix and examination of the sources of supply to the affected area quickly traced the trouble to this particular well which was immediately taken out of service and the distribution system thereby cleared. Investigation revealed that bomb damage refuse had been tipped into the gravel pit and the material was of such a nature that fermentation took place, reducing the *pH* of the water and causing a solution of the iron contained in the accompanying metal refuse. The well-water became contaminated and masses of Crenothrix developed on the chalk face in the depths of the well and fragments of the growth were washed into supply. The trouble was cleared up by treating the gravel pit with excess lime, thus restoring the chemical equilibrium, and the growth in the well was destroyed by chlorination at a dose of 10 parts per million.

Streptomyces and Moulds causing Tastes and Odours in Water

From time to time individual complaints are received from consumers that the water has a musty taste. Investigation usually reveals that the water comes from a tap in a block of large buildings, such as modern flats or large offices. The water certainly tastes musty when first drawn and has a temperature equal to that of the air of the room. After running the tap for a few minutes its temperature drops and there is no taste in the water or it is greatly diminished in degree ; samples of water from a hydrant on the supply main in the street are also free from taste.

The tastes are usually associated with the presence of large numbers of odoriferous moulds and Streptomyces in the water, which can be revealed only by special cultivation methods. It appears that the organisms can multiply in water under certain conditions, such as a raised temperature

associated with infrequent use of the water. Growth possibly occurs on the interior surface of the pipes and the odoriferous substances diffuse into the water.

These organisms are widely dispersed in nature but when conditions are suitable they may multiply, causing food spoilage, etc. Two conditions are necessary for the multiplication of odoriferous Streptomyces in water mains to a sufficient extent to produce musty tastes and odours :

(1) a minimum temperature of 17° C.; and
(2) a sufficient amount of organic material in solution.[34]

Thus growth will not occur in an organically pure chalk well-water, but does occur in a river-derived filtered water. Alkalinity and a high mineral content also favour the growth of Streptomyces. When given the opportunity to proliferate in water, so as to impart an objectionable taste, complaints are to be expected from consumers, who are entitled to receive a water of good physical quality and free from taste and odour as well as being safe to drink. The remedy here lies not with the water undertaking, but in the careful planning of large buildings to avoid long lengths of drinking-water pipes and to ensure that they are insulated by lagging against the heat of the building, or they may be distributed through the building in separate ducts. The problem is not an easy one to solve and is becoming more acute because of the tendency to erect larger buildings and to heat them by methods of central heating which entail the use of hot-water pipe systems for distribution of the heat through the building.

The Part played by the Bacteriologist in Relation to Water

Enough has been written in the Paper to show how important is the part played by bacteria in relation to water, and that this association ramifies throughout all branches of bacteriology—medical, agricultural, and industrial.

Besides the bacteriological considerations of the water itself there is also the question of the health of the workman on the waterworks, a most important aspect because a carrier of waterborne disease may be on one's very own doorstep. Wherever there is a risk of contaminating the water, every new entrant employed by a water undertaking should undergo an examination including enquiries into any history of previous intestinal illness, a blood test and, if necessary, a bacteriological examination of faeces, urine, and duodenal contents. In addition, any waterworks employee whose duties bring him into contact with the water and who is attacked by illness associated with looseness of the bowels should be suspended from such work until he is completely recovered and clinical and bacteriological examinations show that he is free from infection. These examinations should be carried out at laboratories especially equipped for the purpose and staffed by workers qualified to carry out the tests.

The chief value of bacteriology in relation to water is as a delicate indicator of excretal pollution giving warning of its attendant potential danger of waterborne disease. The routine water bacteriologist therefore devotes most of his energy to the discovery of bacterial indices of faecal pollution and the trend today is to rely on frequent sampling, but the tests are made as simple as possible so that opinion on the safety of a water can be given in the minimum period of time. *Bacterium coli* type I, that prime indicator of faecal pollution, can now be identified within 48 hours by modern bacteriological methods and assurance of its absence from a sample is almost certain if there is no sign within 18 hours of the test being set up.

In Great Britain at the present time there are several groups carrying out bacteriological examination of water supplies, including the Water Analyst employed by the undertaking, the Public Health Laboratory Service directed by the Medical Research Council for the Ministry of Health, the Public Analysts, the Consultant Analysts, the Advisory Bacteriological Service of the Ministry of Agriculture and Fisheries, nationalized industries, and commercial undertakings. There may be as many as four of these groups carrying out examinations of the same supply. The necessity for co-operation between these groups on methods and interpretation of results is manifest and from time to time in the past a question has arisen regarding the consistency of results by different analysts. It is a pleasure to record the remarkable closeness of the results obtained when examining samples collected from the same source at the same time by several different sample collectors, bearing in mind the random errors of sampling inherent in all this type of bacteriological work. Much of the success is due to adherence to the procedures of sampling and bacteriological technique laid down in Report 71 of the Ministry of Health on " The Bacteriological Examination of Water Supplies." [31]

Special and more exhaustive bacteriological tests are employed when examining a new source or as a confirmatory procedure, but it should always be borne in mind that the bacteriological examination is only part of the investigation of a source and that little reliance can be placed on the examination of one sample only. The bacteriologist should base his opinion upon a series of samples collected under varying meteorological conditions, and he must also take into account the history of the supply, the details of the topographical inspection, the time of the year with prevailing meteorological conditions and the result of chemical analysis together with geological considerations. Again, this calls for collaboration between the various experts dealing with the provision of water supplies both in the investigation of new sources and in the control of existing supplies. Thus the Water Analyst should consult the Water Engineer and the Medical Officer of Health and work in the closest liaison and harmony with them in order to achieve this common end of providing a safe and adequate water supply for the people.

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The Paper is accompanied by five sheets of diagrams, from which the Figures in the text have been prepared.

Discussion

Mr M. R. James said that the Paper was a very real and useful contribution to the literature of the Institution. It should be one of the principal books of reference to be found in the office of every water engineer. It explained concisely, and in terms which were easily understood by those who were not bacteriologists but mere engineers (who talked in terms of thousandths or even in fractions of a thousandth of an inch whereas the Author referred to microns and millimicrons), the basis of bacteriology in relation to water supply, and it should go a long way to assist water engineers in interpreting the results of water analyses.

When he looked back over a period of about 25 years and remembered the prejudice and suspicion, on the various works with which he had been associated, about the gentlemen with sample jugs, who had asked all sorts of questions and had been regarded as a general nuisance, he felt that much of that could have been dissipated had such a Paper then existed. He recommended all senior water engineers to make a point of drawing the attention of the younger members of their qualified staff to it.

Could the Author confirm the deduction Mr James had drawn that in river-derived waters the bacteria were not spore-bearing ? In London it had for many years been the general practice, as a first line of defence against bacteriological impurity, to pass all river-water through open storage reservoirs and to subject it to the beneficial effects of equalization of quality and natural processes of improvement in the open air and sunlight. He had always understood that by doing that one could reduce the bacterial content of the water by several hundreds and even thousands of times ; that was, the quality of the water on leaving the reservoir would be several hundred, even a thousand, times better from a bacteriological point of view than on entering it. In support of his deduction that the bacteria were not spore-bearing was the point that reliance was placed on terminal chlorination for the final sterilization of the filtrate. He imagined that if they were spore-bearing, chlorination would be useless and there would be very serious epidemics from time to time.

There was another point that he would like the Author to confirm. Might it be assumed that if the water being delivered into supply after final treatment showed negative results for *Bacterium coli* type I, there would be no possibility of the presence of pathogenic organisms of a dangerous type because the latter were so much less hardy than the *Bacterium coli* type I ? The organisms found growing on the jute packing of pump glands would appear to be harmless in themselves, but their presence in samples could give a misleading impression to those not acquainted with the proper interpretation of bacteriological analyses.

In that connexion, the Author had modestly omitted to record in the Paper the work he had done, in conjunction with the Distribution Engineer of the Metropolitan Water Board, in setting up an organization for the routine and carefully controlled sterilization of jute for pump glands and of yarn for the jointing of pipes. Perhaps he could incorporate in his reply some notes on that work, for they would be valuable to the Institution. That process of regular sterilization had gone a long way to remove the doubts and irritations that used to exist in the minds of the people who had to do the work in the trench in the street and maintain the supply, for on many occasions in the past, positive results had been obtained from samples from mains under repair, and the re-charging of the mains and bringing them back into service had had to be delayed until it was certain that no harmful bacteria were present. The bacteria had a masking effect.

When routine sterilization of yarn took place it was very unlikely that

positive results would be obtained after the usual proper and careful disinfection of the mains which had been repaired.

In connexion with the question of tastes apparently associated with moulds, he suggested that, although some of the objectionable tastes were caused by the water being raised to the temperature of rooms in heated buildings, most of the complaints received probably arose from the fact that cold water pipes used for drinking purposes were housed in ducts alongside pipes carrying hot water for the supply of baths and lavatory basins. In the latter, the temperature might easily reach 80° C., whereas space temperatures seldom exceeded 40° C.

No reference appeared to have been made in the Paper regarding the necessity for dechlorinating samples taken at works which were not provided with contact reservoirs. It was an important point, because it ensured that samples taken immediately before delivery into the mains gave a true picture of the condition of the water on leaving the works. That was particularly necessary when there were houses in the neighbourhood of the works which were supplied within a few minutes of the water leaving the works and there was no adequate period of contact to enable the chlorine to do its work.

Table 2 would have been more convincing if it had been continued for several weeks beyond 17—one wondered what the *Bacterium coli* count might have been after that. It might be that the Author saw the figure of 25, rang up the resident distribution engineer in some distress and told him that there was something wrong, and that it was immediately remedied.

With reference to *Figs 2* and *4*, perhaps the Author would have been able better to emphasize the point that he made (that the reduction of ammonia in water passing through rapid sand filters was adversely affected by temperature) if he had also shown graphs of the water temperature. On one of the slides shown by the Author there was a temperature chart; it would be an advantage if such a chart could be incorporated in the Paper. There were many who had every reason to remember the arctic conditions of the winter of 1946–47, but if, as had been suggested, the Paper was to be used by water engineers of the future, for many years to come there would be many who were too young to remember exactly what those conditions were and others who were not then born, and might not even now be born, but who would, he hoped, have the advantage of reading it.

Dr L. A. Allen observed that the Author had stressed the significance of the faecal bacteria, particularly *Bacterium coli* type I and to a lesser extent *Streptococcus faecalis* and *Clostridium welchii*, as indicators of pollution in the examination of water. The main source of those organisms was sewage, but they might gain entrance to bodies of water in small numbers or in special circumstances in comparatively large numbers from other sources. It was of interest to consider the possible sources of pollution and the numbers of faecal bacteria which they might sustain.

In soil the coli-aerogenes group was widely distributed. According to

Taylor,[35] the counts in British soils ranged from nil to more than 100 per gramme and *Bacterium coli* type I was the predominant member. Soil drainage was therefore a potential carrier of those organisms and the numbers would be increased by manuring of the soil. Drainage from farmyards constituted a source of large numbers of faecal bacteria of animal origin.

The leaves and stems of plants supported a varied flora of which the coli-aerogenes group formed part. The numbers of those organisms on the growing plant were normally small, but when the plants were cut and piled in a heap or subjected to a process of fermentation, as in the ripening of silage or the retting of flax, numbers might increase very considerably. In retting liquor, for example, Allen [36] found that *Bacterium coli* type I sometimes attained numbers of ten million per millilitre. Thousands of gallons of that liquid might be discharged daily from a large rettery and unless it received adequate treatment it represented a potentially serious source of faecal bacteria, though the pollution was not, in fact, faecal in character.

Effluents discharged to inland waterways had to conform to a chemical standard of purity, but no bacteriological standard had been set and, although efficient treatment at a sewage works incidentally removed a large proportion of the bacteria originally present in the crude sewage, very large numbers remained after treatment and were discharged with the final effluent. A study of the effect of treatment on the numbers of faecal bacteria in sewage had been made at two sewage works, from which samples had been taken regularly for several months. One was a rural works receiving only domestic sewage and the other was a large urban works where the sewage contained a large proportion of industrial wastes. The average bacterial counts in the effluents, after treatment in percolating filters and humus tanks, were as shown in Table 5.

TABLE 5

	Bacterial counts per millilitre of effluent	
	Bacterium coli	*Streptococcus faecalis*
Small rural works	3,400	900
Large urban works	4,900	400

It was evident that a sewage works discharging numbers of bacteria of that order into a river would greatly increase the bacterial load on the river,

[35] C. B. Taylor, "*Coli-aerogenes* bacteria in soils." *J. Hyg. Camb.*, vol. 49 (1951), p. 162.

[36] L. A. Allen, "Recent Development in the microbiology of retting. Part I.—The Microbial flora in different systems of retting." *J. Soc. Chem. Ind.* vol. 65 (1946) p. 365.

unless the flow of the stream was very large compared with the volume of effluent discharged.

Many single houses on the banks of lakes and streams treated the household sewage in a septic tank installation, and the effluent was discharged intermittently. Results of observations recently made on five such plants over a period of several months were given in Table 6. The septic tanks differed in design in each installation but the effluent from each one was treated in a small automatic percolating filter.

TABLE 6

(Overall values for five plants)

	Bacterium coli per millilitre		*Streptococcus faecalis* per millilitre	
	Septic tank	Final effluent	Septic tank	Final effluent
Maximum	625,000	59,000	66,000	4,200
Minimum	14,000	255	0	0
Average	114,000	10,400	8,400	630

The effect of sewage effluent on the numbers of faecal bacteria in the river to which it was discharged had been studied some years ago on the River Avon.[37] The sewage of Coventry was treated at two works, one of which discharged its effluent direct to the Avon at Baginton; the other works discharged its effluent to the River Stowe, a tributary of the Avon, which reached the main stream about 5 miles below Baginton. Counts of *Bacterium coli* in samples taken from the River Avon at various points along its course showed that the immediate effect of the effluents was to increase the numbers of those organisms very considerably. The counts decreased very quickly, however, as the discharged effluent flowed downstream, and in the course of 3 or 4 miles most of the organisms had been removed from the water, either by death or by sedimentation. Similar results were found for *Streptococcus faecalis*. It was concluded that, at a point several miles below the effluent outfall, the effect of a minor source of pollution, such as the discharge from a septic tank, in the near neighbourhood might be more serious than the residual effect of the major source of pollution upstream.

The rate at which bacterial counts decreased below a source of pollution varied considerably in different streams, for reasons not yet entirely understood. In a tributary of the River Colne, consisting almost entirely of sewage effluent, there was a decrease of about 75 per cent in the numbers

[37] Department of Scientific and Industrial Research. Report of the Director of Water Pollution Research, 1939–1945. H.M.S.O., 1947.

of *Bacterium coli* over a distance of about half a mile (see Table 7). In saline waters, *Streptococcus faecalis* was able to survive better than *Bacterium coli*, so for such samples it might be a more important indicator of pollution than it was for samples of, say, drinking water.

TABLE 7.—DIMINUTION IN NUMBERS OF *Bacterium coli* IN EFFLUENT STREAM WITH INCREASING DISTANCE FROM SOURCE OF POLLUTION

Distance (yards)	0	150	350	600	850
Count (per millilitre of water)	20,700	14,100	9,400	8,200	5,400

The length of time for which organisms survived in water depended upon a number of factors such as *pH* value, content of organic matter, degree of oxygenation, and the presence or absence of substances with a bactericidal action. *Bacterium coli*, for example, would actually grow in water containing as little as 0·3 part per million of nutrient organic matter; *Streptococcus faecalis* required about 50 times that concentration before it could grow. Both organisms, however, might survive for long periods in water where the concentration of nutrient material was not sufficient to allow growth. Thus the rate of disappearance of those organisms was much more rapid in some streams than in others.

Recently the ability of faecal bacteria to survive in the waters of the Avon and the Sowe was studied. Samples of water from different points were taken to the laboratory and all bacteria were removed by passing the water through a Seitz filter. Each sample was inoculated with a mixed culture of *Bacterium coli* (6 pure strains) and *Streptococcus faecalis* (7 pure strains). Counts were made at intervals during storage of the samples at 20° C., and the results were shown in Table 8. In three of the four samples studied both *Bacterium coli* and *Streptoccocus faecalis* died fairly rapidly. In the remaining sample, *Bacterium coli* actually increased considerably in numbers and *Streptococcus faecalis* died only very slowly. Water taken from the River Sowe for that sample evidently contained sufficient nutriment to permit the growth of *Bacterium coli* and greatly to retard the rate of death of *Streptococcus faecalis*. In a sample taken a hundred yards or so farther down the same stream, after it had received a discharge of sewage effluent, the water was comparatively lethal to both organisms; that suggested that the sewage effluent contained bacteriophage, or antibiotic or other bactericidal substances.

Mr F. N. Green said that the Author had referred to the high sensitivity of bacteriological analyses. That was strikingly borne out by a case of pollution in a chalk well with which he had recently been connected. The number of *Bacterium coli* present had been 1,800 per 100 millilitre, which the Author would no doubt agree was a gross pollution. Whilst that pollution was present—and it had continued over several

TABLE 8.—SURVIVAL OR GROWTH OF MIXED SUSPENSIONS OF *Bacterium coli* TYPE I (6 STRAINS) AND *Streptococcus faecalis* (7 STRAINS) IN SEITZ-FILTERED RIVER-WATER DURING STORAGE AT 20° C.

(Counts are expressed as percentages of the counts of the freshly inoculated river-water)

Period of storage : days	Source of sample of river water			
	River Avon		River Sowe	
	Above Baginton outfall	Below Baginton outfall	Above Finham outfall	Below Finham outfall
	Bacterium coli type I			
0	100	100	100	100
1	15	13	290	77
2	3·5	5·0	—	9·4
3	1·0	1·9	8,200	5·0
6	—	—	8,200	3·2
7	—	—	8,300	2·7
	Streptococcus faecalis			
0	100	100	100	100
1	2·5	2·5	74	49
2	0·6	0·6	76	0·2
3	0·1	0·4	64	0·1
6	—	—	49	—
7	—	—	52	—

weeks—there had been no increase in free or albuminoid ammonia or the oxygen absorbed. A chemical analysis would not have shown any change in the water, yet from the bacteriological point of view it was undoubtedly grossly polluted.

It was suggested on p. 408 that organisms of the Salmonella group lived harmlessly in animals, being consequently passed on, and on p. 412 it was stated again that animals could pass on pollution ; Mr Green understood that those organisms were pathogenic bacteria, and therefore harmful. The report of the Ministry of Health on Gathering Grounds, published in 1948, stated in paragraph 24 that if a test to distinguish between animal and human faeces could be devised, then the main objection to pollution by animals would disappear. The report implied that the only trouble that arose from animal pollution was the masking or absorption of chlorine during treatment. Sir William Savage, representing the Society of Medical Officers of Health, had said in his evidence that " The Pathogenic organisms which were water-borne do not affect animals so such animal-borne excreta is not a means of spreading them." There appeared to be a conflict of opinion and it would be interesting to hear the Author's views on the point.

Could the Author give evidence to confirm his statement that the I.A.C. group tended to live longer in the external environment than *Bacterium coli* type I ? Mr Green felt that it was probably true. If there were evidence he would be grateful for some information. The Author had said that there was little difference between the resistance of *Bacterium coli* type I and I.A.C. organisms to chlorination, but Mr Green wondered whether their differing lives in the external environment would also give them a differing resistance to chlorination.

He did not dispute the acceptance of *Bacterium coli* as an indicator of pollution, for it was the simplest test for checking a water supply quickly, but were they right in accepting *Bacterium coli* as the check on chlorination ? He felt that it ought not to be assumed that just because it was the indicator of pollution it must also be the indicator of efficient sterilization.

He had found the Author's reference to the aerogenes-coli ratio in Liverpool water rather difficult to understand. It was stated that Gray had found the aerogenes-coli ratio relatively low in recently fouled water but the ratio was rapidly reversed by storage, and that the proportion of aerogenes to coli in the Liverpool water supply was relatively high and increased with storage. He found that ambiguous, and would be grateful if the Author could clarify the point.

With regard to the theory that the presence of an I.A.C. group might indicate that *Bacterium coli* type I would follow, he wondered whether a service reservoir was the right part of an undertaking on which to build a theory of that type. Where was the pollution coming from ? If external pollution was entering the reservoir other than through the pipes feeding it, surely the theory need not hold. The only evidence on the point, so far as Brighton was concerned, partly supported the Author's theory. Over several years tests had been made at service reservoirs. The presence of *Bacterium coli* in the reservoir had always been preceded by the I.A.C. group, but that had happened on only a few occasions. On the other hand, the appearance of I.A.C. organisms in the reservoir on many occasions had not been followed by *Bacterium coli*. On the evidence available, it could not be assumed that if I.A.C. were present, *Bacterium coli* must follow.

With reference to the Author's example of the chalk borehole, no mention was made of rainfall. Surely that was a very large factor in the pollution of wells. Mr Green had tried many times to relate pollution of a well to rainfall and had failed to get a true relationship, but he was still convinced that the two were inter-related.

With reference to the well with gross pollution to which reference had been made, the pollution had appeared on three separate occasions, each lasting several weeks. In the first case, the pollution was almost entirely due to *Bacterium coli*. In the second case, the I.A.C. organisms accounted for more than 90 per cent of the pollution, *Bacterium coli* being almost absent. In the third case, the I.A.C. and the *Bacterium coli* organisms appeared in the ratio of 2 to 1. In the first condition, the water in the

well was at its lowest level, in the second the water was rising, and in the third the water was falling. He felt certain that the water-level conditions, related as they were to rainfall, had some connexion with the various ratios of the two groups. He believed that the results could be used to show that the two groups of organisms had a different life. In the first case the well was low and in the second the water was rising. The interpretation of that was that the pollution entered the well when it was at its lowest and when the water rose it washed the sides of the fissures supplying the well and took off the I.A.C. groups, which had probably outlived the *Bacterium coli*.

With regard to the general treatment of water, the Author had suggested that one could not get efficient chlorination in the presence of ammonia. All agreed on that, but it might be misread by some as indicating that the Author did not agree with chloramination using post-ammoniation. With the latter system, a very marked difference in the results of reservoir analyses had been found.

The Author had referred to the necessity for the frequent testing of water from pumping stations. Mr Green wondered if that was the best system. Brighton sampled water from all stations daily, but the water was actually taken from the district supplied. That meant that one was checking not only the raw water and the treatment but also the reservoirs and the distribution system. Station samples were taken less frequently; did the Author think that there was any weakness in that system? If daily samples were to be taken at several points the amount of work in the laboratory would grow enormously.

Mr Green was amazed at the lack of bacteriological supervision in many waterworks—even relatively large undertakings were without laboratories.

A lead from the Author to reduce the work involved in testing workmen would have been useful. The collection of urine and faeces samples was difficult and the Widal test result was unreliable after T.A.B.—Typhoid, Para-Typhoid Type A and Para-Typhoid Type B inoculations. If the Vi serum test could be relied upon the amount of work involved would be greatly reduced.

Mr F. J. R. Taylor, dealing with bacterial activity in power stations, said that the problems facing the British Electricity Authority in preparing water for boiler feed resulted in requirements in many ways more stringent than those faced by the water companies. There was, however, a common problem in the removal of ammonia. Most of the impurities could be removed by softening followed by distillation in an evaporator, but ammonia, especially in large amounts, was still something of a problem.

Thus, he was very interested in the Author's work on the removal of ammonia from water supplies by bacterial treatment. The B.E.A. had not so far applied that process to feed-water preparation, but it was applying it (and had done so unconsciously in the past) to river-water wherever it had cooling towers. Recent analytical results had shown that a large cooling tower was capable of destroying nearly 1 cwt of ammonia per hour

if the water which was cooled was grossly polluted. His opinion was that it was not unreasonable to suggest that with a highly polluted river-water —say, one with about 15 parts per million of ammoniacal nitrogen—used for make-up to a tower system, the concentration in such a tower system might be of the order of $\times$ 1·7 and the ammonia would be expected to be concentrated to 25 parts per million. But he believed that it would be reduced to about 0·3 part per million, owing to the bacterial activity in the water.

It was of some interest to note in passing that if cooling water was taken from a river which was short of oxygen and passed through a cooling-tower system, the oxygen content would be substantially increased. That arose from the aerating effect of the tower.

Besides the bacteria which purified water, there were what might be called bacterial diseases of power plants. There were some micro-organisms which did their best to destroy the timber packing in the cooling towers. There were other micro-organisms which grew on the condenser tubes and cut down the rate of heat transfer so that chlorination or other measures were required to remove the bacterial growths.

Mr F. E. Bruce observed that the Author had said that outbreaks of waterborne dysentery and waterborne enteritis—he was not sure whether anything lower in the scale than those was included—were rare but undoubtedly occurred. He presumed that the Author meant that they were rare in comparison with the numbers of such diseases caused through contaminated food. There were several reasons why that should be so. The organisms could not survive in water for such a great length of time as they could in food, where they would probably find suitable nourishment and could multiply. There were also other causes, and in the case of the milder intestinal diseases there were probably many cases which occurred but were not reported as such. Waterborne dysentery and gastro-enteritis were probably more common than was generally recognized.

The importance of that might not be immediately apparent, because such diseases were usually of a mild character. But if such cases were at all frequent, it must mean considerable loss of man-days through people being away from work for a day or two at a time. Although that situation might not arise in the cities and towns of Great Britain, it might be a serious matter in rural areas and in countries where the water supply was not given such good attention.

Referring to the Chicago Hotel dysentery outbreak of 1933, Mr Bruce pointed out that that was a case of amoebic dysentery and not bacillary dysentery as seemed to be implied on p. 407. Perhaps the Author would indicate whether the other outbreaks described were of bacillary dysentery or amoebic dysentery.

With regard to the Chicago case, there was a point which was relevant to what he had said about some cases not being reported and investigated as cases of dysentery. It had been stated that the diagnosis of amoebic

dysentery had not been made correctly in a very large number of cases in the Chicago outbreak. Until it had been recognized as an outbreak of amoebic dysentery it seemed that various cases were mistaken for such things as appendicitis, colitis, ulcerative colitis, cholecystitis, hepatitis, malignancy of the intestines, duodenal disease, tuberculosis of the intestines, pleural effusion, typhoid fever, and ulcer of the stomach. He had no wish to undermine the respect in which they held their medical colleagues, but the Author, as a medical man, might have some scathing remarks to make on the efficiency of diagnosis. The point was that there might be many cases, which were not recognized as dysentery and were not investigated, which might be caused by waterborne organisms.

Mr Bruce outlined a problem, not strictly in the field of water supply but indirectly related to it. It was that of a town which had hitherto treated its sewage purely by land irrigation, and had decided to build a modern treatment plant and to develop housing on the land which had been used as a sewage farm. He presumed that some period should elapse before the housing development began on the site and that the land should first be put to agricultural use for a year or two. From what he knew from the published information on the survival of organisms in soil, most of the intestinal organisms would probably die out within a matter of months, but he wondered what would be the case with the gas-gangrene bacillus, *Clostridium welchii*, which could form spores and survive for very long periods. Could the Author indicate whether that would be a serious menace and what period should be allowed before the land was used for any other purpose than agriculture ?

Mr M. C. West said that he knew the Author to be a cautious man, for he was one of the engineers whom the Author kept waiting, while samples of water, collected from the works, were slowly cooked at blood heat for 18 hours ; but would he say whether there was any prospect in the forseeable future of being able to pass samples through a suitable filter, put them under an electron microscope, and promptly tell the engineers whether the water could be put into the supply ? Was that an engineer's pipe-dream ?

Mr Bernard Whitteron asked the Author to state what weight should be attached to the dangers of dust and airborne infection.

Mr R. C. S. Walters said that the Author had stressed the importance of keeping filters warm. Was it to be concluded that, where practicable, the ordinary rapid gravity sand filter should be roofed rather than left in the open, as so many of those filters were ? It was a debatable point among engineers.

Mr J. B. K. Ley asked if the survival of bacteria was influenced by the slightly higher forms of life in water, such as algae and protozoa.

Mr C. D. C. Braine said that the Paper would be even more valuable as a handbook if the Author could add a little to it about bacteria in sea-water and allied problems.

**** **Mr F. P. Dath** referred to his experience a few years previously, at a certain water undertaking where he had discovered that even elementary precautions were not being observed. At one pumping station, sterilization had been effected by injecting a chloros solution into an open contact tank. The pump used for that was often breaking down, and occasionally, when the necessary spare parts had not been ordered in time, long periods occurred during which no chloros at all had been put in. Any suggestion to cover up the tank had been met with some scorn and derision, but he did, however, manage to have the tank cleared out and amongst other things uprooted two reeds about 2 feet 6 inches in length, which had grown up from the bottom of the tank. For three summers, when it could be surreptitiously accomplished, the accumulation of flies and dirt had been skimmed off the top of the tank. There was no main drainage at that station and the cesspool, although covered, was only a short distance from the tank.

Mr Dath had confirmed his suspicion that some of the mains had been above the hydraulic gradient, by reason of a haphazard switching about of booster pumps, when a consumer had said that, since installation of a certain pump, no water was available after 4 o'clock—only a sucking noise on opening the taps.

Mr Dath had managed to sterilize a few mains before they were put into supply when he had been able to learn of their readiness for service. Some of those mains had been lying in ditches with a farm effluent trickling through them. He had also been successful in getting a small brick shed built to house a portable gas plant and he believed that the tank had been subsequently covered up.

He was surprised that the conditions under which that undertaking operated had not been responsible for serious outbreaks of waterborne diseases.

The Author, in reply, said that it was always his endeavour to achieve the fullest co-operation between the water engineers and the water analysts. That was the only way in which they could do their work and achieve success in that aspect of public health, which it was their responsibility to keep in a satisfactory state, namely, the purity of the drinking water supplies.

He appeared not to have brought out sufficiently that the spore-bearing organisms really represented only a proportion, and a resistant minority, of the whole of the bacterial world. Spore-bearing organisms were divided into two main groups. There were the aerobic ones, those living in the air. One of those, anthrax, was a serious one from the point of view of human disease. The others were harmless organisms which lived in the air and soil—including the hay bacillus which lived in vegetation —and they were not reduced by storage. In the case of water supplies,

**** This contribution was submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

such as rivers, large numbers of spore-forming organisms were present, particularly in the winter; they were resistant to chlorine and some survived through the purification system and represented a high proportion of the plate counts in river-derived waters. They were not of epidemiological importance.

The other group consisted of those that lived in the absence of oxygen, the anaerobic spore-bearing organisms, including the tetanus bacillus and the gas-gangrene bacillus. Again, the gas-gangrene bacillus, or *Clostridium welchii*, was not reduced very much by storage and was also chlorine-resistant.

Fortunately, the pathogenic intestinal organisms—typhoid, paratyphoid, and dysentery—were non-spore-forming and were known as vegetative organisms. It was those which were so considerably reduced in reservoirs on storage as a result of the unnatural environment. Thus many of the pathogenic organisms, being vegetative forms, were killed by the effects of storage and the remainder were removed by filtration or destroyed by disinfection with chlorine, ozone, etc.

In taking *Bacterium coli* type I as the indicator of faecal pollution, they were relying on an assumption which began with Frankland in the 1890's and it had stood the test so far. The assumption was that typhoid and other vegetative bacteria could not live any longer than *Bacterium coli* type I and, if it could be demonstrated that the latter micro-organisms had disappeared, it was safe to assume that the pathogenic intestinal organisms had also disappeared.

Mr James had mentioned jute yarn and the work which had been done in the 10 years since 1943. The reason why there was a problem to solve was the fact that jute was infected with micro-organisms, and if the material was shown to contain *Bacterium coli* type I it might also contain pathogenic organisms. The Author had grown typhoid, paratyphoid, food poisoning, and dysentery organisms on jute. An American observer had stated that an extract of jute yarn was almost as good as agar as a medium for cultivating bacteria.

Although jute yarn had never been proved to be the cause of water-borne disease, it did not follow that that was not so, but it had been definitely established that, by its very nature, the material was grossly infected with bacteria, including intestinal types. After all the trouble that was now taken to purify water for domestic use, up to the highest level of bacterial quality, it seemed illogical to introduce a packing into a pump and into a pipe joint that brought the bacterial quality of, say, a river-derived water, back to its original low level, or changed a pure well-water into a grossly polluted one. Owing to the nature of the material, even heavy doses of chlorine were ineffective in sterilizing yarn, and it was decided to sterilize the material in bulk, by other means, before it was issued. The process involved the use of an organic mercurial compound and was most rigidly controlled by laboratory tests. As a result of the use of sterilized yarn

followed by chlorine disinfection of mains, the quality of water samples collected from repaired mains was now as high as that in the general distribution system.[38, 39]

He had attempted to imply in the Paper that tastes which were acquired in distributing pipes in large premises were caused by local conditions of heat together with stagnation due to intermittent use and, it if were not for those local conditions of heat, the tastes would not appear. A temperature as low as 17° C. was sufficient for the organisms to multiply and give rise to the taste in water containing organic matter.

He had not dealt with the dechlorination of water samples because he had tried to avoid all technicalities. He agreed that any samples collected for bacteriological analysis—be they taken from a main leading to a service reservoir, from anywhere in the distribution system, or from any point after the water had been chlorinated—should be dechlorinated at the time of sampling, in order to obtain as accurate an estimation as possible of the bacterial content of the water.

With reference to Table 2, it was correct to assume that he had taken some action when the *Bacterium coli* count reached 25 per 100 millilitres. The reservoir had been taken out of supply and the water went " down the drain."

There were no actual temperature figures available in respect of *Figs 2* and *4*, but he had mentioned in the text that, at the time of the very large decrease in ammonia (11th March, 1947), the temperature of the water was only 1° C. It was only after that experience that the relation between temperature and ammonia oxidation in the filters had been studied more carefully, and *Figs 3* were included in the Paper to show the effect of a fall in temperature.

He was obliged to Dr Allen for having enumerated the sources of the faecal organism in sites not strictly faecal. Dr Allen had brought out the point that sewage effluents, passed as purified, still contained a very high proportion of bacteria, and had said that one was often inclined to be led astray by a suggestion that a sewage effluent was bacteriologically a satisfactory liquid, even although it had not been finally sterilized with chlorine. That was far from the case. Although a great many bacteria were removed, that was only relative, and, from the bacteriologist's and the public health point of view, the number removed had very little significance in relation to what was required to be done to the water in order to make it fit for a domestic water supply.

The graphs of the River Avon illustrated the marvellous self-purifying effect still present in many British rivers. That was still the case with the River Thames, and the River Lee also had a considerable rejuvenating effect by the removal of undesirable bacteria. Rivers recovered rapidly from successive sewage effluents and, if they were not too frequent, a

[38] References 38 to 44 are given on p. 447.

river such as the River Thames could cope with the discharges from the large communities along its banks.

The Author was interested in Mr Green's experience of a high *Bacterium coli* count in a chalk-well water—1,800 per 100 millilitres—without any chemical alteration. The Metropolitan Water Board had had a similar experience about 3 years previously. The first warning had been a very high *Bacterium coli* count—1,600 per 100 millilitres—without any evidence of falling off in the chemical quality. The next further evidence of pollution was an increase in the demand of the water for chlorine.

The question about animal pollution on gathering grounds was a little difficult to answer. He considered that the report quoted dealt only with animal pollution in large reservoirs, the implication being that the occasional pollution that could occur in a large lake or reservoir would be so considerably diluted and destroyed by prolonged storage that it would have very little epidemiological significance. It should be remembered that every safeguard was made to prevent the direct access of grazing animals to the reservoirs and to its feeder channels.

With reference to the Liverpool water, the Author said that Gray was trying to show that, when the water first entered the reservoir, the number of *Bacterium coli* in relation to the number of *Bacterium aerogenes* was very high, but as a result of storage, the ratio became reversed. Gray concluded that *Bacterium aerogenes* strains were much more resistant to the elements and that the change resulted from the different rates of dying off. As the Author had said, the whole question of the faecal significance of I.A.C. organisms was still debatable and he had felt very hesitant in introducing the topic at all. In putting forward two aspects of the importance of the I.A.C. group he had hoped that it would lead to some discussion, and it had done so.

There was no doubt that I.A.C. organisms survived longer than *Bacterium coli* type I outside the human and animal intestine, and there was much evidence that their habitat was primarily non-faecal. That did not mean, however, that I.A.C. organisms were more resistant than *Bacterium coli* type I to chlorine ; any differences observed in that direction were of no significance from a practical point of view. As stated in the Paper, a water that had been satisfactorily chlorinated should contain no coliform organisms. On the occasions when I.A.C. appeared in a chlorinated sample there might be several explanations ; too small a dose of chlorine to destroy vegetative organisms ; the organisms existing in some focus, such as jute yarn, whose nature was such that the chlorine could not penetrate ; or, the organisms had entered the water at some point after chlorination.

With regard to Mr Green's experience, his I.A.C. organisms were evidence of past pollution or more serious pollution to come, as the Author had shown in the case of the chalk borehole supply where rainfall had caused an effect within 24 hours. In that case, he chose a particular

month because he wanted to show a period when there was a high proportion of I.A.C. organisms in relation to *Bacterium coli*. During many months of the year, *Fig. 1* would look " black all over," but he wanted to show that the intermediates, *Bacterium aerogenes* and the *Bacterium cloacae* were forerunners of worse pollution and had to be taken seriously. He was dealing with the odd samples when *Bacterium coli* type I was absent. If samples had been collected only on those particular days—which worked out, roughly, as weekly sampling—one would have obtained the I.A.C. organisms with no *Bacterium coli* type I at all. He had given that as an example. On the other hand, he had tried to show that there were occasions when even higher I.A.C. counts might mean no pollution at all, in the faecal sense, as was the case with the dug well where the organisms had been traced to decomposing wood.

Thus the presence of I.A.C. organisms in the absence of *Bacterium coli* type I might mean one of two possibilities, which could only be confirmed by subsequent samples. First, a low I.A.C. count might be followed by the appearance of *Bacterium coli* type I and the only conclusion to be arrived at within our present knowledge was that faecal contamination existed; secondly, if no *Bacterium coli* type I appeared in subsequent samples and, in particular, if the I.A.C. increased in numbers, then those organisms were probably derived from an innocuous source and were of no epidemiological significance.

He had not been trying to detract from the efficacy of chloramination, a treatment which had a definite place in waterworks practice. A very good variation of the method was to have post-ammoniation, to add ammonia after the chlorine, in order to establish a residual of chlorine in the distribution system.

He agreed that samples in the district checked both the treatment of the raw water and also the quality of the water in the service reservoirs and mains, but he considered that station samples should be taken as frequently as district samples, in order to exert a closer control on the treatment of the water. Perhaps the best compromise in Mr Green's area would be station samples and district samples on alternate days.

Tests relating to workmen should be carried out only in laboratories which were especially equipped and the results should be interpreted by persons who had been especially trained. Such persons would have had experience in interpreting the results of blood tests upon men who had had T.A.B. inoculations and would look for substances in the blood, such as the Vi agglutinin, which were not much affected by previous inoculation. The Author emphasized that the Widal test was only a sorting test and that bacteriological tests should be carried out if the blood test was suspicious.[40] Every reasonable practicable precaution must be taken. The Ministry had laid down certain regulations which must be followed as closely as possible.* For example, it would be considered that everything in reason

* See ref. 5, p. 428.

had been done if, in the case of a suspicious blood test without any other suspicious circumstances, three bacteriological examinations, spread over a month, had been negative.

He had been interested in Mr Taylor's account of the removal of ammonia in cooling towers. He presumed that the organisms were seeded there from the polluted water supply used in the cooling process, and the large amount of ammonia removed probably resulted from the open method, thus producing ample oxygen for the chemical processes involved.

There was evidence in less-well-aerated systems that the oxygen was somewhat reduced by the organisms to obtain oxygen to change the ammonia into nitrite and nitrate.

He referred Mr Bruce, regarding a pocket book on bacteriology for engineers, to the chapter dealing with bacteriology in the Manual of British Waterworks Practice.

He still contended that waterborne enteritis was rare compared with the number of outbreaks caused by food, because the organisms multiplied in food and massive doses were necessary to cause infection. Many cases of food poisoning were not bacterial, but caused by toxins, soluble substances excreted by bacteria ; that was another mode of attack on human beings.

He was fully aware that the Chicago dysentery outbreak was of the amoebic type, but outbreaks to which he had referred were all bacillary. The diagnoses made in the Chicago epidemic merely showed that the medical men in America, as elsewhere, were only human and liable to " miss " the early cases in an epidemic of an infrequently occurring disease, before the situation was fully realized.

With reference to the projected housing developments of land formerly used for sewage disposal, giving it over to agriculture for a year should be sufficient. He imagined that it would take at least that time for the land to become sufficiently stabilized to enable the engineering and housing people to put up houses. The survival of *Clostridium welchii* did not come into the question. Where there had been contamination there would be *Clostridium welchii* ; that organism existed all the time, wherever land had been manured. It was one of the normal inhabitants of the intestine but, so long as it stayed there, it did not matter at all. Gardeners would have a very " rough " time if their gardens did not contain substances which had *Clostridium welchii* in them. There was always the risk of infection by soil contaminating a deep cut or wound, but the risk was no greater from the type of land in question than from any other cultivated soil.

With reference to the " pipe-dream," he could give Mr West a ray of hope. Methods were being developed which might result in an answer regarding the purity of water samples being furnished within 12 or 14 hours. But that involved awkward times of the day ; it meant that analysts would have to stay up late and probably awaken engineers to

give them the results. Eighteen hours was a very convenient period. By commencing the tests at 4 p.m., they were able to telephone an opinion on quality to the engineer at 10 a.m. the next morning. They were now confronted by a dilemma, for, by speeding up the methods they might " land " themselves in administrative difficulties.

Mr Whitteron had raised a rather difficult point. Service reservoirs should have the smallest possible openings. Ventilators should be so designed as to prevent dust entering. Dust would probably not be harmful unless it had some specific micro-organism in it. The effect of dust would be estimated by carrying out total bacterial counts. The answer was to reduce as much as possible any opportunity for dust to enter by careful planning in the construction of the reservoir.

In reply to Mr Walters he would very much welcome the roofing of rapid filters. Not only would it keep them a little warmer in winter and so tend to maintain the ammonia oxidation process, but, much more important, it would prevent the heavy algal growths produced by the effect of sunlight on the supernatant water. With roofing, however, sufficient ventilation would have to be provided.

Mr Ley had asked whether higher forms of life had any effect on bacteria. The Author believed that much of the killing effect and self-purification effect of rivers and on the surface of filter beds was caused by protozoa and other small animals and plants. He did not believe that they left typhoid alone and chose harmless bacteria. Much of the improvement in bacterial quality of water, in fact, resulted from the ingestion of bacteria by those slightly higher forms of life.

He believed that Mr Braine had had in mind the survival of bacteria in sea-water and the possible risk of bathing in sewage-polluted water. Dealing with the first point, a study of the literature on the longevity of bacteria in sea-water indicated that survival was much the same as in fresh water. Experiments on the survival of typhoid organisms revealed that in different experiments 85–79 per cent of the organisms died out in the first 2 days, but survivors could be demonstrated after 32 days.[41] A variety of food-poisoning bacteria had been isolated from the tidal River Plate and it was concluded that the organisms could survive sufficiently long to be a danger to health.[42] Bacterial spores of the *Clostridium welchii* type could survive for months in the sea.[43] An examination of the bacterial flora in the intestines of fish revealed a high content of intestinal organisms in fish caught near the shore where the sea-water was polluted with sewage, but the intestinal contents of fish caught in the open sea were virtually sterile.[44]

There was, therefore, a risk of infection from bathing in sewage-polluted water, and a bathing resort, quite apart from aesthetic considerations, had to reduce sewage pollution of its sea and beaches to the minimum in order to avoid infection and consequent loss of its reputation. That did not mean that sea-water had to be up to the same standard of

bacterial quality as drinking water, but it should be of such a reasonably high order that provided a margin of safety.

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40. E. F. W. Mackenzie and E. W. Taylor, " A Study of the Vi-agglutination Test for the Detection of Typhoid Carriers." *J. Hyg. Camb.*, vol. 44 (1945), p. 31.
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42. O. A. Peso, R. H. Leiguardia, and J. C. Kempny, " Research into the pathogenic intestinal bacteria in the waters of the River Plate." *Obras San. de la Nacion*, vol. 13 (1949), p. 101.
43. B. C. Callame, " Survival of anaerobic bacteria in sea water." *Ann. Inst. Pasteur*, vol. 82 (1952), p. 377.
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The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—Sec. I.C.E.

STRUCTURAL AND BUILDING ENGINEERING DIVISION MEETING

7 May, 1953

Professor A. J. S. Pippard, Member, Chairman of the Division, in the Chair

Four Introductory Notes were presented to initiate a discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

"Would the Strength Grading of Ordinary Portland Cement be a Contribution to Structural Economy?"

Introductory Note No. 1, by Frederick Sidney Snow, M.I.C.E.

Introductory Note No. 2, by Frederick Measham Lea, C.B.E., D.Sc.

Introductory Note No. 3, by Leonard John Murdock, M.Sc.(Eng.), Ph.D., A.M.I.C.E.

Introductory Note No. 4, by Edward Burke, M.Sc.(Eng.), A.M.I.C.E.

Introductory Note No. 1
by Mr Snow

It is well known that there are a great number of factors affecting the strength of concrete. It is also an undoubted fact that the 28-day strength of cement from different works, even though conditions appear to be as nearly identical as possible, ranges from 3,000 to 5,000 lb. per square inch.[1]

So that too much time is not wasted by subsequent speakers, it should be pointed out that, among the many reasons given by the cement manufacturers for lack of uniformity in their products, one is that some of their works are out-of-date and so far they have not been able to obtain building licences or materials to modernize them. This point has been placed before the Ministry of Works by the Author of this Note, as the representative of the Institution on the Advisory Council on Building Research

[1] W. L. Scott, W. H. Glanville, and F. G. Thomas, "Explanatory Handbook on the B.S. Code of Practice for Reinforced Concrete," Concrete Publications Ltd, 1950. *See* Fig. 1A, "Age-Strength Curves for Concrete."

and Development, and the Ministry admit that whilst they consider the manufacture of cement vitally important, the facts, as stated, are substantially correct.

It has been further pointed out by the cement manufacturers that the quality of coal supplied to them is, in many cases, such that it does not enable them to manufacture a cement of high quality; again, this statement has been corroborated by the Ministry of Fuel and Power.

The cement manufacturers have also stressed that closer supervision of the manufacture of cement through all its stages would assist in the production of a more uniform product, and the Ministry of labour have agreed that they are not in a position to supply the necessary labour for this particular purpose.

It would appear, therefore, that the manufacturers have a complete answer on these particular points. However, this does not absolve them from the fact that they have one standard price for all cements, irrespective of quality, which, on the face of it, is hard to comprehend. No other industry would be permitted to sell an inferior article at the same price as an article of superior quality.

It is a well-known fact that practically all cements will give a strength of concrete in excess of British Standards requirements, but if there is greater uniformity in the manufacture of cement, with the consequent grading of quality, then it is obvious that if the highest-quality cement is used, the size and thickness of structural members could be correspondingly reduced; the Paper [2] by Graham and Martin on Heathrow provides evidence of this.

The suggestion is put forward, therefore, that where a low-quality concrete is required for such purposes as drainage-pipe surrounds, filling behind retaining walls, housing, etc., it should be possible to purchase a low-grade cement at a reduced price. In this connexion, the Author would point out that by far the greatest quantity of cement produced and used falls within this category.[3]

Similarly, where the concrete has to comply with a British Standard, a structural grade of cement should be obtainable and, where the highest-quality concrete is called for—for instance, in runways, prestressed members, etc.—a high-quality cement should be procurable with possibly a commensurate increase in the price.

It may well be that the cement manufacturers' argument is that, owing to rationalization, the difficulties created in accountancy are such that the overall price of cement would be increased. Be that as it may,

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[2] G. Graham and F. R. Martin, "Heathrow. The Construction of High-grade Quality Concrete Paving for Modern Transport Aircraft." J. Instn Civ. Engrs, vol. 26, No. 6, p. 117 (April 1946).

[3] F. N. Sparkes, "Control of Variations in Quality of Concrete and its Effect on Mix Proportions." Technical Paper No. 10, Reinforced Concr. Assocn, 1949. *See* Table 4, "Summary of Factors Contributing to Variation in Concrete Strength."

the Author of this Note strongly feels that the cement manufacturers should solve the economic problems involved in this matter and that any increase in cost should *not* be passed on to the civil and building industries.

Another relevant point is that the cost of raw materials for the manufacture of cement is not the same in every given instance, and this is a condition which presumably in a competitive world cannot be overcome, nor is it desirable that it should be.

It is as well at this stage to point out that many large contractors are in a position to insist upon having cement from works where the quality is known to be high, and they are able to purchase this commodity at the same figure as an inferior cement which the small contractor perforce has to buy.

It may be argued that the cement manufacturer leaves the strength of the concrete to the designer of the mix. For instance, where a 12 : 1 concrete has been specified for drain surrounds, if a high-quality cement is forthcoming, this mix might well be increased to 18 : 1 or even 24 : 1. In the present state of affairs, therefore, how can such matters be left to the individual designer ?

There is no doubt that the cement industry in Great Britain has some of the finest chemists in the world ; its cement is cheaper than that produced in other countries and to this extent the British are far in advance of any other nation. The work carried out in the research department of the cement manufacturers is second to none, and whilst it is possible to appreciate fully the difficulties and problems, some logical explanation is still required as to why it is necessary to pay 84*s*. per ton for a 3,000-lb.-per-square-inch cement, and the same price for a 5,000-lb.-per-square-inch cement.

Introductory Note No. 2 *
by Dr Lea

For many years it has been the practice at the Building Research Station to check each batch of cement received at the Station for compliance with British Standard 12, and also to test concrete cubes made with standard aggregates in accordance with the requirements of Codes of Practice (for example, C.P. 114) for " Preliminary cube tests." The results of these tests are of interest in considering the desirability of introducing new strength grades for Portland cement.

Although about 140 batches of cement have been tested, only a limited number of brands of cement have been used and the results may, therefore, not be fully representative of all marketed cements. In particular, a wider survey would probably indicate greater variability than has been

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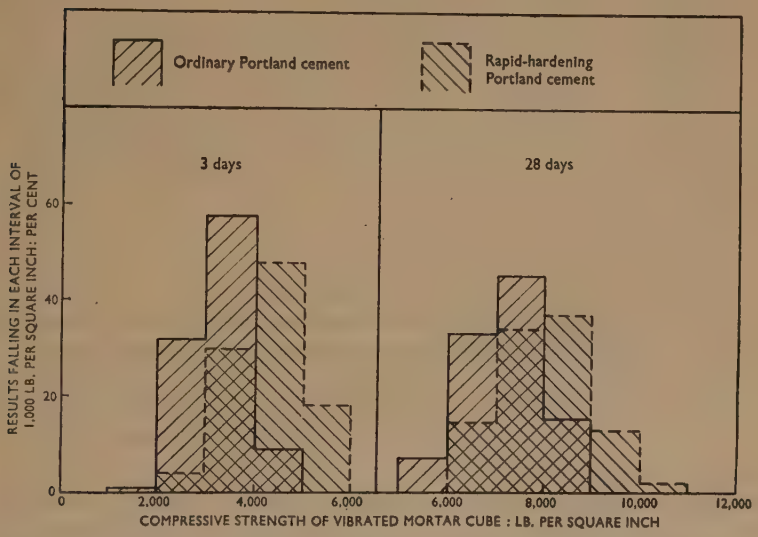
found in the B.R.S. tests. Nevertheless, the data presented in this Note are thought to be a useful guide to the strength characteristics of cements and their influence on concrete strengths.

Results of Strength Tests on Cements

In Tables 1 to 3 the results are summarized of both tensile tests on cement briquettes and compressive tests on standard vibrated mortar cubes. In the first two Tables the percentages of results falling within each of a series of actual strength ranges are given, whereas in Table 3

Fig. 1



Comparison of the Compressive Strengths of Vibrated Mortar Cubes made with Ordinary and Rapid-Hardening Portland Cements

the percentages are related to ranges of the ratio of actual strength to the minimum strength prescribed by B.S.12 for the particular type of cement, method of test, and age. The results for the compressive strength of vibrated mortar cubes shown in Table 2 are also illustrated in the histograms of *Fig. 1*, which indicate that about half the ordinary and rapid-hardening Portland cements overlap in strength.

The Tables also indicate the wide variation in cement strength at ages up to 28 days, and there is obviously a need for careful consideration of this variation in order to obtain the most efficient and economical use of cement in building and structural engineering works.

TABLE 1.—DISTRIBUTION OF TENSILE STRENGTHS OF CEMENT (B.R.S. DATA)

Tensile strength: lb. per square inch	Percentage falling within each range							
	Ordinary Portland cement (81 cements tested)				Rapid-hardening Portland cement (56 cements tested)			
	1-day	3-day	7-day	28-day	1-day	3-day	7-day	28-day
0–100	—	—	—	—	—	—	—	—
100–200	11	—	—	—	4	—	—	—
200–300	69	1	—	—	18	—	—	—
300–400	19	37	6	—	55	5	—	—
400–500	1	57	59	22	23	50	25	5
500–600	—	5	34	64	—	45	62	50
600–700	—	—	1	14	—	—	13	41
700–800	—	—	—	—	—	—	—	4
Approximate mean .	260	420	480	540	350	490	540	590
B.S. minimum . .	—	300	375	—	300	450	—	—

TABLE 2.—DISTRIBUTION OF COMPRESSIVE STRENGTHS OF CEMENT MORTARS IN B.S. VIBRATED MORTAR CUBES (B.R.S. DATA)

Compressive strength: lb. per square inch	Percentage falling within each range							
	Ordinary Portland cement				Rapid-hardening Portland cement			
	1-day	3-day	7-day	28-day	1-day	3-day	7-day	28-day
0– 1,000	37	—	—	—	13	—	—	—
1,000– 2,000	63	1	—	—	66	—	—	—
2,000– 3,000	—	32	1	—	21	4	—	—
3,000– 4,000	—	58	5	—	—	30	—	—
4,000– 5,000	—	9	56	—	—	48	13	—
5,000– 6,000	—	—	36	7	—	18	46	—
6,000– 7,000	—	—	2	33	—	—	29	14
7,000– 8,000	—	—	—	45	—	—	12	34
8,000– 9,000	—	—	—	15	—	—	—	37
9,000–10,000	—	—	—	—	—	—	—	13
10,000–11,000	—	—	—	—	—	—	—	2
Approximate mean .	1,100	3,200	4,800	7,200	1,600	4,300	5,900	8,000
B.S. minimum . .	—	1,600	2,500	—	1,600	3,500	—	—

TABLE 3.—DISTRIBUTION OF CEMENT STRENGTHS RELATIVE TO B.S.12 MINIMUM REQUIREMENTS (B.R.S. DATA, STANDARD TESTS)

Range of ratio of actual strength to B.S.12 minimum	Percentage falling within each range							
	Ordinary Portland cement				Rapid-hardening Portland cement			
	Tensile		Compressive		Tensile		Compressive	
	3-day	7-day	3-day	7-day	1-day	3-day	1-day	3-day
0·5 –0·75	—	—	—	—	4	—	18	2
0·75–1·0	1	4	—	—	18	25	21	11
1·0 –1·25	19	38	1	1	37	71	39	41
1·25–1·5	59	54	4	2	37	4	11	35
1·5 –1·75	20	4	19	19	4	—	9	11
1·75–2·0	1	—	16	40	—	—	2	—
2·0 –2·25	—	—	34	24	—	—	—	—
2·25–2·5	—	—	17	13	—	—	—	—
2·5 –2·75	—	—	7	1	—	—	—	—
2·75–3·0	—	—	2	—	—	—	—	—

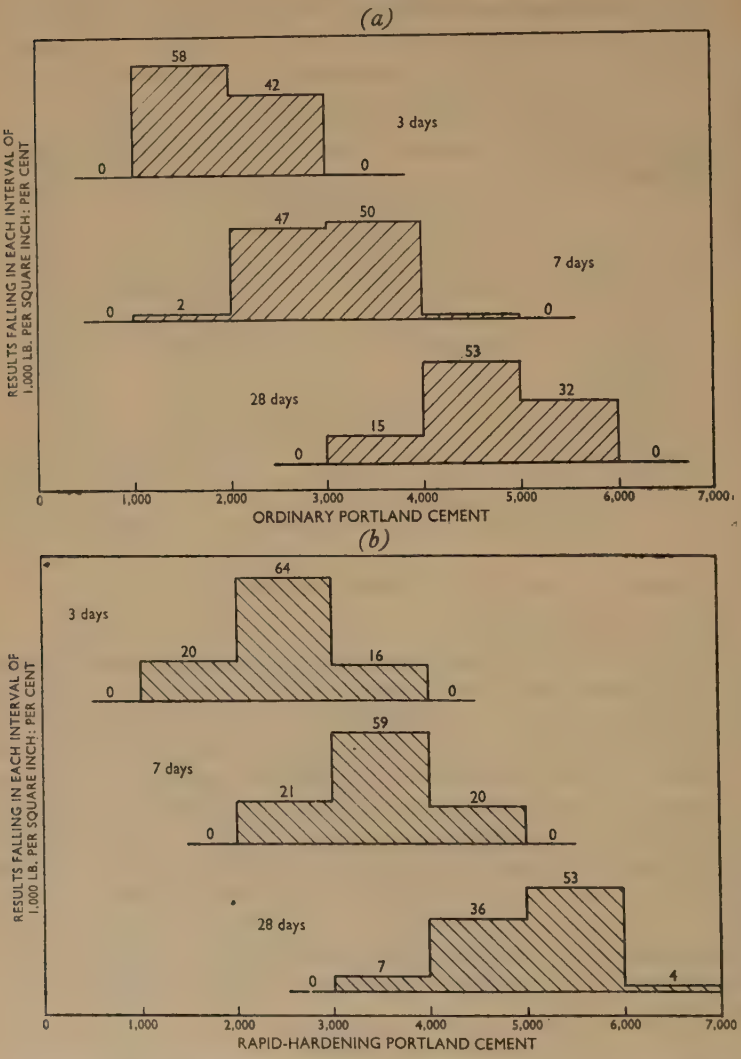
DEPENDENCE OF CONCRETE STRENGTH ON CEMENT STRENGTH

The wide variations that often occur in the strength of the concrete cubes made on constructional works arise from so many factors that only in favourable cases is it possible to isolate the effect of the cement alone. In the standard tests at the B.R.S., however, concrete cubes are made with each batch of cement and a standard aggregate consisting of washed uncrushed river gravel and sand of controlled grading. The concrete proportions (by weight) are 1 : 2 : 4, 0·60 and the preparation, storage, and testing follow the recommendations of C.P. 114. The distributions of concrete strengths are given in *Figs 2* for 119 samples using ordinary Portland cement, and 71 samples using rapid-hardening Portland cement. Comparison of the individual concrete strengths with the mortar compressive strength of the cements used for making the concrete showed that there was a close correlation between the two strengths.

VARIABILITY OF CONCRETE STRENGTH AND ITS RELATIONSHIP TO DESIGN

The permissible stresses in concrete, used in structural engineering design, are based on the minimum " works strength " of the concrete at an age of 28 days. There is, of course, no measure of the actual minimum strength that may occur and the specified minimum may be regarded as the value for which there is only a small probability—of, say, 1 per cent

Figs 2



COMPRESSIVE STRENGTHS OF CONCRETE MADE WITH STANDARD AGGREGATES AND MIX PROPORTIONS (1 : 2 : 4, 0·60)

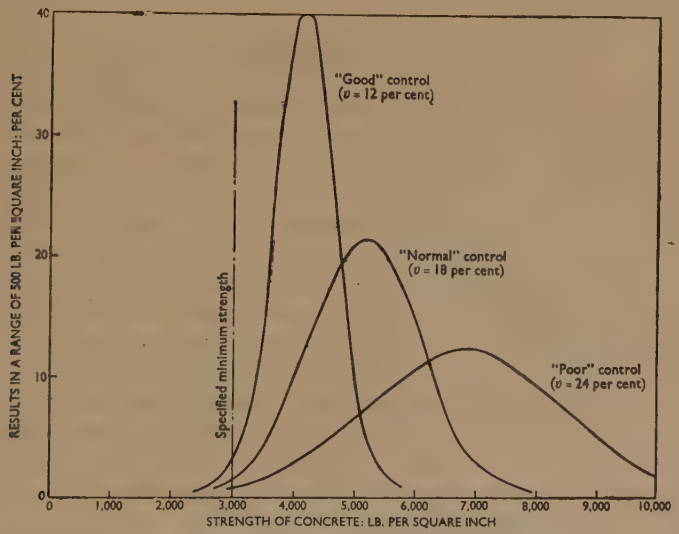
or less—that lower strengths will occur for the particular concrete proportions and degree of quality control.

This method of dealing with the variability of concrete strength means that a considerable proportion of concrete used in structures is very much stronger than necessary to fulfil the intended purpose, simply in order to

ensure that no individual sample will fall below some fixed value. Cement is thus being wasted and this is particularly so when the variability of the concrete strength (as expressed, for example, by the coefficient of variation) is high. This variability of concrete strength arises both from that of the cement and from the degree of control exercised in the concrete-making. The variability arising from poor control will much outweigh that arising from the cement itself, but with good control the latter becomes an important cause of the variation in the concrete strength.

An example indicating the variability of concrete strength for three degrees of control is shown in *Fig. 3*.

Fig. 3



CONCRETE STRENGTHS FOR THREE DEGREES OF CONTROL

Distribution is such that not more than 1 per cent of the results are lower than 3,000 lb. per square inch in each case. v denotes coefficient of variation

Investigation of concrete strengths on typical jobs shows that the coefficient of variation for the works strength of concrete could be reduced to about 10–12 per cent with very-good-quality control, but that a coefficient as high as 27 per cent can occur with poor control. In order to ensure that not more than one set of cubes in a hundred falls below the specified minimum, the average strength must have a much higher value if the control is poor, as indicated in Table 4.

It is interesting to note that the coefficient of variation for the 28-day-strength results of standard concrete tests at the B.R.S. (see *Figs. 2*) was 13·5 per cent for both ordinary and rapid-hardening Portland concrete. The lower coefficients for " Good " control in practice evidently requires

TABLE 4

Degree of control	Coefficient of variation (v) : per cent	Necessary average strength divided by minimum specified strength
Good	12	1·4
Normal	18	1·7
Poor	24	2·3

a much stricter control on the variability of cement than mere compliance with B.S.12.

POSSIBLE STRENGTH GRADINGS OF CEMENT

The object of any further sub-division of ordinary and rapid-hardening Portland cement into, say, three grades would be to provide materials more closely related to the requirements of the user and so enable a more economical use to be made of cement. It may raise economic and supply problems but these will be left for discussion by the manufacturers.

Since permissible concrete stresses are normally linked with 28-day strengths, they do not in themselves demand any particular strength of a cement at earlier ages. The need for this arises from other causes. Thus, for speed in testing, it is necessary that tests be made at earlier ages and used as an indication of the strength that will be developed later. Again, certain minimum rates of development of strength are required for constructional convenience—for example, early removal of shuttering, or as a safeguard in cold weather, etc. Further, certain types of work—such as prestressed concrete—demand a rapid development of strength as an economic necessity in manufacture.

If three grades of cement were to be introduced they might be classified as follows :—

Grade 1.—For all general building work, such as site concrete, haunching, brickwork, etc.

Grade 2.—For structural work in plain or reinforced concrete.

Grade 3.—For special reinforced-concrete work, such as shell structures, and for prestressed concrete.

The requirements for Grade 1 could remain the same as for ordinary cement at present, and those for Grades 2 and 3 could be respectively lower and higher than is at present required for rapid-hardening cement. The Grade 3 would presumably only be demanded where high strength at early ages was essential. The use of Grade 2 cement for structural work would enable the designer to rely on somewhat leaner mixes to achieve a specified 28-day concrete strength than is practicable with ordinary

cement at present, since it would have a higher guaranteed minimum strength.

With these assumptions as a basis, the strength gradings shown in Table 5 might be considered.

TABLE 5.—PROPOSED STRENGTH GRADINGS

Cement	Minimum compressive strength of vibrated mortar cube : lb. per square inch			
	1-day	3-day	7-day	28-day
Grade 1	—	1,600	2,500	(4,000)
Grade 2	—	2,500	4,000	(5,000)
Grade 3	(1,600)	4,000	5,500	(7,000)
Present ordinary	—	1,600	2,500	—
Present rapid-hardening	1,600	3,500	—	—

If it were desired to introduce a 1-day-strength requirement, this need be applied only to Grade 3.

The data presented in this Note indicate that all cements now being marketed should satisfy the requirements for Grade 1. All rapid hardening Portland cements and a high proportion of ordinary Portland cements would satisfy the requirements of Grade 2. More than half the rapid-hardening Portland cements and some of the ordinary Portland cements now available would satisfy the requirements of Grade 3. Thus it should be possible to get these three grades of cement by selection from present manufacture without the need for developing new and stronger cements. This selection would have the additional effect that the variability of strength within each grade would be smaller than the present variability. Greater uniformity may well be as important as an enhanced minimum strength in allowing improved economy of material in structural design.

Introductory Note No. 3
by Dr Murdock

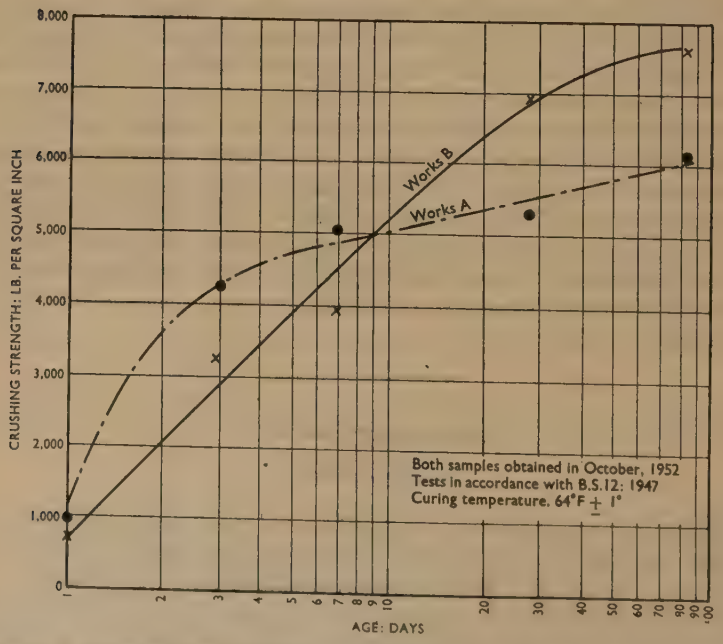
PORTLAND cement is used by the building and civil engineering industries in large quantities for a multitude of purposes, quite apart from its use in concrete. For many of these purposes, including brick mortar and renderings, no great strength is required, and any cement complying with the minimum requirements of the present British Standards is acceptable. Further, much of the concrete produced does not require to have a very

high strength—for instance, mass concrete generally, oversite concrete for houses, and sub-bases to roads. It follows that a considerable proportion of the output of cement could have relatively low strength characteristics, that is, with a much smaller margin in excess of the existing British Standards requirements than is common with most present-day British cements, provided it could be produced and delivered to the site at a low price. Such a cement would not necessarily be inferior and, in fact, might possess advantages in, say, lower drying shrinkage, and slower heat-generation.

If several grades of cement were introduced, it appears unlikely that every works would be able to produce all grades, and consumers might have to face long and costly hauls if they insisted on a certain grade which was not in production locally. In the United States of America, although specifications exist for a number of grades of cements, it is very difficult to obtain any other than the two common grades, unless the requirements are large enough to take the whole output of at least one kiln.

Apart from these practical difficulties, there is the question of the method of grading cements for strength. As illustrated in *Fig. 4*, in

Fig. 4

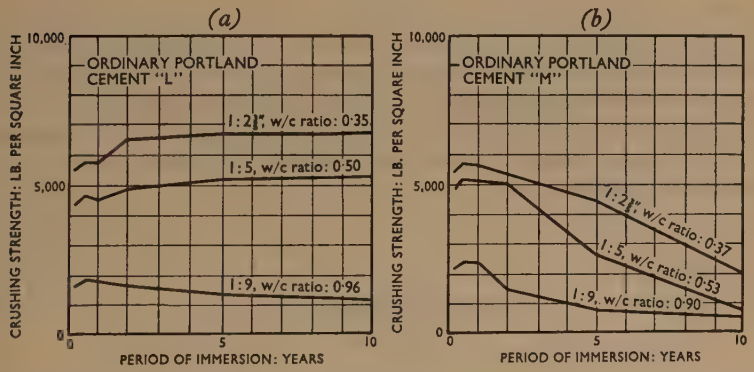


Increase in Mortar-Cube Strength with Age of Two Samples of Ordinary Portland Cement

which the crushing strengths of vibrated mortar cubes made from two samples taken from deliveries of cement from different sources are compared, ordinary Portland cements do not all harden at the same rate, so that the grading would possibly differ according to the age at which the crushing strength was decided. The grading system appears likely to become cumbersome if provision is made to cover such differences. Presumably, provision would also be necessary for cements at present classified as rapid-hardening Portland cements.

A strength-grading system does not provide any guide to the other properties of cements, in which differences are found between those from various works and possibly even between cement from the same works from time to time. Sulphate resistance may be taken as an example, since it is a property which is often of considerable importance, and may even give a clue to durability under other conditions where chemical attack or erosion is a problem. Thus, differences in resistance to chemical attack by sea-water between two cements of very much the same strength at 1 year are shown in *Figs 5*. Cement " L " appears to have been highly

Figs 5



Resistance to Sea-Water Attack of Two Samples of Ordinary Portland Cement (Lea and Davey, 1949)

resistant, even when compared with high-alumina cement, whereas cement " M " was much less resistant, but it must be remembered that these results apply only to the particular samples tested, and further investigation of the variation in resistance of any particular cement from time to time is very necessary.

More attention needs to be given to the influence of the cement on durability, permeability, and other useful properties, and it is undesirable to place too much emphasis on strength requirements by the introduction of a grading system based on strengths alone.

In view of the disadvantages of a strength-grading system, in that it

centres interest on only one property of the cement, whilst it might lead to supply difficulties and an increase in cost, particularly for work not requiring a high-strength cement, it is suggested as an alternative that :—

(1) The present British Standard for ordinary Portland cement is retained as providing a minimum standard, but with crushing-strength requirements included as obligatory instead of optional as at present. If the vibrated-mortar-cube test is not generally acceptable, it is essential that another test for crushing strength, not affected by human factors, should be included.

(2) The cement manufacturers should publish data, for the information of consumers, giving the general characteristics of the cement they produce. This specification would follow the lines adopted in other industries—for example, the electrical, mechanical, and motor industries—and would include information on crushing-strength characteristics, chemical composition, fineness, soundness, shrinkage properties, sulphate resistance, and durability.

The cement manufacturers would then be able to adjust output to the local demands to obtain maximum economy, whilst the consumers would have all the necessary information before them, which would enable them to choose between accepting the local supply if this was of suitable quality, or, if it would be more economical or more satisfactory for the particular usage, hauling a cement with more desirable properties from a more distant manufacturer.

Introductory Note No. 4
by Mr Burke

WHETHER the strength grading of cement is, from an engineering viewpoint, desirable, is best discussed by the other speakers. This Note will be confined to the implications of the simplest grading possible—namely, the addition of another grade of ordinary Portland cement to the existing range of cements.

It is well known that cement frequently far exceeds the British Standards requirements. However, owing to the variations at and between works and in testing, the manufacturers can guarantee only to provide cement passing this specification. The variation between works arises largely from the different raw materials and coal used at different works. The variation at a particular works is caused largely by the varying ash content of the coal. Uncontrolled changes in the composition of the raw materials fed to the kilns have been almost completely eliminated, but the

variation in the coal ash is at present outside the control of the cement industry.

The variation in the coal ash at a works may be, during quite a short period, as much as 20 to 30 per cent, and is quite commonly 5 to 10 per cent. That this is a serious matter is clear when it is realized that a 10-per-cent variation in coal ash can lead to a 6-per-cent change in the lime saturation factor which, as is well known, is one of the determining factors in rate of hardening.

Investigations are proceeding into methods of overcoming this difficulty, but the best that can be done at the present time is to exercise ingenuity in blending, so that at the worst the cement does not fall below the standard specification.

There is a popular misconception that rapid-hardening Portland cements are made at the most modern and efficient works while slow-hardening (so-called low quality) Portland cements come from out-of-date works. This is not so. The question is complex but, as mentioned above, determining factors are the nature of the raw materials and the quality of the coal. In some cases, the slower-hardening cements are more expensive to manufacture.

Investigations are proceeding to find a way to reduce the variation in rate of hardening caused by the major factors already mentioned. Assuming, therefore, that there will come a time when present efforts are successful and that the variation can be largely reduced, would it then be practicable to have two grades of ordinary Portland cement ?

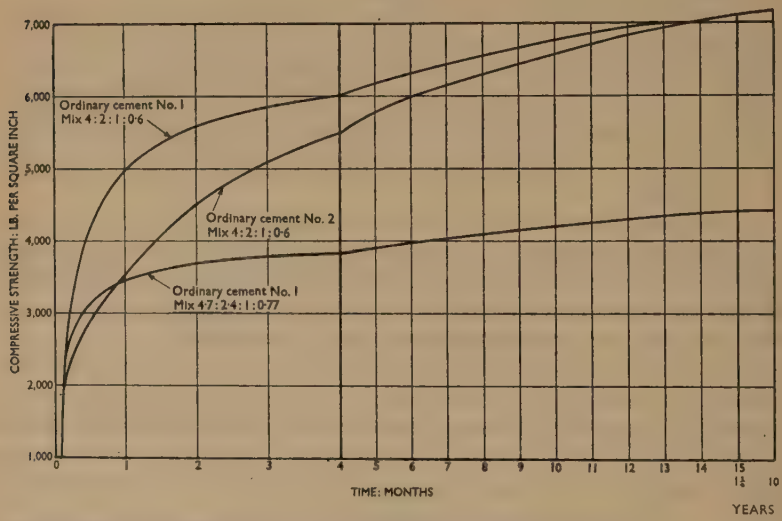
In most cases a choice of several grades of cement benefits the convenience of construction rather than the ultimate strength of the job. There is evidence that, for comparable mixes made with different grades of cement, the wide differences in early strength disappear with increasing age so that after, say, 1 year there is little to choose between the resulting concretes. This is also true of different batches of the same cement whose early variability tends to be reduced in time. Thus, whilst variability undoubtedly exists and the industry is endeavouring to reduce it, it should be recognized as mainly a short-term matter and of critical importance only to those engineers for whom a stated early strength is essential. However, the evidence so far is that the variation in cement quality in Great Britain is no greater than, and probably less than, that found abroad.

Assuming that downward swings have been eliminated, the addition of a new grade of Portland cement would clearly necessitate the re-classifying of existing ordinary Portland cement as high-grade and the introduction of a new lower grade of cement. Is this what engineers have in mind ? If so, who would want the lower grade, bearing in mind that it would not be cheaper than the higher grade ? It should also be remembered that increasing the number of grades of cement produced means extra plant and storage capacity or increased stoppages in changing over. A greatly

increased demand for cement might counterbalance this effect but it should be remembered that the cement industry is now on full production.

It may be argued from an economic standpoint that cement giving high early strength permits economy in mix design. It is true that a weaker mix made with rapid-hardening Portland cement may give an early strength equal to or greater than that of a richer mix made with a slow-hardening cement. But *Fig. 6* shows how ultimately the slow-hardening Portland cement mix will be much stronger and, being richer, it should have much greater durability, that is, resistance to frost, chemical attack, and so on.

Fig. 6



Relationship Between 28-Day and Final Compressive Strength of Concrete

Mention of early strengths raises the question of cement- and concrete-testing. One reason why the manufacturer makes his cement well above specification, apart from the downward swings caused by coal-ash variation, is that there is considerable variation between testing laboratories. To allow for this and for some degree of aeration between testing at the works and testing of the site sample, the cement must be made with a strength at all ages at least 30 per cent above specification.

The question of a suitable test for cement has exercised the minds of the manufacturers for some time. For many years they have based their manufacture on a concrete-test. It is only in recent years, however, that this test has been developed to a stage where it is likely to be useful for inclusion in a standard specification by formulating it so as to give reasonably reproducible results in all properly equipped laboratories. This test

is at present under consideration by a Committee of the British Standards Institute.

To sum up, there are practical difficulties in introducing a new grade of Portland cement. A first step would be to find ways of overcoming the present uncontrollable factor of coal quality. The next would be to reduce the variation in cement-testing because the difference in strength between ordinary Portland cement as at present manufactured and that of rapid-hardening cement is not great enough to make the introduction of another grade reasonably practicable.

Variability in strength is, however, essentially a short-term problem. Nevertheless the industry intends to reduce it, and it is felt that production of a single high-quality cement of low variability, instead of two grades as at present suggested, would be a useful and, what is more, a practicable contribution to structural economy.

Discussion

Dr Oscar Faber said that the subject covered by the Paper could be divided into two parts: (1) how desirable was it to have a British Standard for stronger cement; and (2) how far was it possible for the cement industry to comply with the request if it were made today? With regard to (2), obviously a great many difficulties would have to be overcome. For instance, the ash content of coal delivered was known to range from between 20 and 30 per cent to between 4 and 5 per cent, and it was not surprising that there were differences in the resultant product. It seemed that the only way to modify that situation was through an approach to the Coal Board, but how far it might be possible for them to vary their practice he could not say.

Next, there was the difficulty that some works were more up-to-date than others. How far was it practicable to obtain a certain quality of cement from distant works when there were works close at hand producing cement of a different quality? There would be various commercial difficulties in having different grades, beyond visualization or solution.

It seemed, therefore, that the second part of the problem had to be left to the cement industry, to weigh up the difficulties and see whether it was possible to do what, on some grounds, seemed desirable.

As to the first part, Dr Faber was Chairman of the Code of Practice Committee for reinforced concrete—that Code was at present under revision—and he had proposed that in the new specification, the stresses could be raised by 10 per cent, as well as the cube strength required to justify that. Practically everyone of a responsible character to whom he had proposed that had turned it down on the ground that compliance with the present British Standard could not guarantee concretes any stronger than were

indicated in the present Code cube strength. It was well known that most of the cements would give strengths 60 to 100 per cent above those figures, but the cements were much better than those specified in the British Standard—and any cement which satisfied the British Standard had to be acceptable to the Code. It had to be considered what cube strength could be relied upon with a cement which only just passed the British Standard.

Great Britain, Dr Faber said, made the cheapest and the best cement in the world. It seemed rather a pity that we had a standard specification which did not advertise that fact but rather suggested the opposite—the British Standard was so low that nearly all cements gave results either 50 or 100 per cent better. Nor could advantage be taken of that high quality, because any cement that passed the low British Standard had to be accepted both by the Code and by the user.

When the argument about raising the agreed stresses had been made, many authoritative facts were gathered together by very important bodies. It had been found that in attempting to produce a concrete (with a 1 : 2 : 4 mix) which satisfied the requirements of 3,000 lb. per square inch, the great bulk of cements greatly exceeded that—the average being 4,408 and the maximum 8,500. Test cubes falling short of the requirements of 3,000 lb. per square inch were 8·15 per cent—an appreciable percentage. Dr Faber wondered what happened when one of that percentage was received. Was the concrete taken out? Was the job started again? Apparently a meeting was held on the job, a lot of people talked for a certain length of time, and then they went home and nothing more happened!

A similar situation had arisen in respect of the 1 : $1\frac{1}{2}$: 3 mix, where the requirement was 3,750 lb. per square inch, and where the percentage which failed had been 6·75. Those were all 28-day cubes, but with 7-day cubes the number which did not pass was very much greater.

The Committee had tried to make up its mind what should be done. There were many causes of those enormous discrepancies—control, water/cement ratio, and other factors. But the principal cause was variations in the cement. *Fig. 7* showed the result of a large number of tests on 3-day mortar cubes, in accordance with British Standard 12.

The maximum strength was 5,250 lb. per square inch and practically nothing failed to reach 1,600 lb. per square inch. Nearly every cube exceeded 2,500, which was the British Standard requirement multiplied by 1·56. *Fig. 8* showed the results of the 7-day tests where the British Standard requirement was 2,500 lb. per square inch.

Very few test cubes failed to pass. If the British Standard requirement was raised by 60 per cent, represented by the 4,000 lb. per square inch in *Fig. 8*, practically everything would pass. That was a matter of interest to the Code of Practice Committee, who were anxious to justify the use of higher stresses which obviously made for economy, because a stronger cement meant a stronger concrete.

It seemed, therefore, that there was a good case for asking for a British

Fig. 7

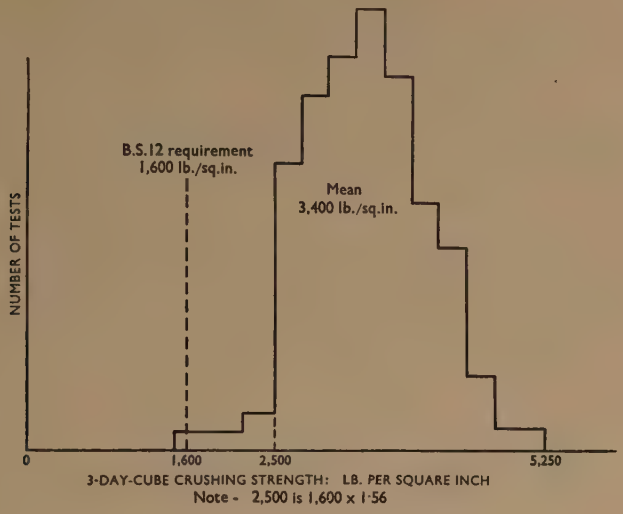
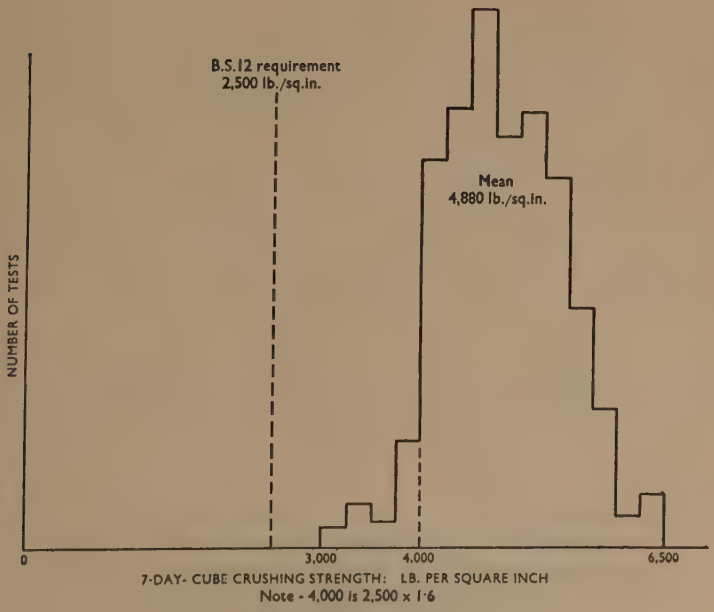

B.S.12 requirement
1,600 lb./sq.in.
Mean
3,400 lb./sq.in.
NUMBER OF TESTS
0
1,600
2,500
5,250
3-DAY-CUBE CRUSHING STRENGTH: LB. PER SQUARE INCH
Note - 2,500 is 1,600 x 1·56

Fig. 8


B.S.12 requirement
2,500 lb./sq.in.
Mean
4,880 lb./sq.in.
NUMBER OF TESTS
0
3,000
4,000
6,500
7-DAY- CUBE CRUSHING STRENGTH: LB. PER SQUARE INCH
Note - 4,000 is 2,500 x 1·6

Standard specification with strengths approximately 60 per cent higher than at present. Indeed, the Committee found that only with a cement having a strength approximately 60 per cent higher than present British Standard requirements could they guarantee the cube strengths already specified in the Code of Practice.

Mr A. B. Harman said that Mr Snow had mentioned a 12 : 1 mix for drain surrounds with a possible increase to 18 : 1 or even 24 : 1 if higher quality cement were forthcoming. Surely it was in that type of concrete that the quality of the cement was least critical and control of the mix most difficult ? Dr Lea had said " The variability arising from poor control will much outweigh that arising from the cement itself, but with good control the latter becomes an important cause of the variation in the concrete strength." Mr Harman said he would be surprised if a change in grade of cement would allow such a change in mix proportions as was suggested. When Mr Snow referred to higher quality, presumably he meant higher strength ; and what of the other qualities ? Whatever the grade of ordinary Portland cement, that mix, used in drain surrounds, was apt to be a little dangerous, particularly with regard to sulphate attack, which was not as uncommon as some people supposed. There was also the danger of thinking that the new grade of cement was better than the old so that less care was needed in making the concrete.

Mr Burke had said that the determining factors in the strength of ordinary Portland cement were the nature of the raw materials and the quality of the coal. That was true to a great extent, but Mr Harman quoted an interesting case which did not bear it out. In one contract, where there was a good control, it had been found that the cube strengths from a number of different mixers were gradually drifting downwards. The cement was tested, and some mortar cubes were below, and some just above, the British Standard requirement. The matter had been taken up with the manufacturer, who had reduced his output by 12½ per cent and calcined the cement harder in the manufacturing process. There was an immediate improvement in the quality and strength of the cement. To some extent that suggested that the solution was often in the manufacturer's hands.

There was already a form of strength grading in the division between ordinary Portland cement and rapid-hardening Portland cement, the difference being in the higher temperature to which rapid-hardening cement was burnt and the greater fineness to which it was ground.

From Table 3 of Dr Lea's Note, one learnt that 39 per cent of the rapid-hardening Portland cements failed to satisfy the 1-day compressive-strength requirement of the British Standard. That figure did not greatly surprise Mr Harman, but was it not remarkable that if it was taken as a guide to the state of affairs throughout Great Britain, then those who bought rapid-hardening Portland cement were paying an extra 7*s*. 6*d*. per

ton for something which, on an average, they were getting only two-thirds of the time ?

It was true that the grading of ordinary Portland cement could not strictly be compared with the present division between ordinary and rapid-hardening Portland cement, but the figures at least made him wonder how reliable would be the different grades of ordinary Portland cement, if they were introduced, particularly in relation to the possibility of one works maintaining a supply of consistent grades within reasonably close limits. The designer had to have confidence that the new grades would be maintained.

Mr Burke had said that in any case the lower grade would be no cheaper than the better grade—which did not seem to strengthen the case for grading ordinary Portland cement. Without being pessimistic, Mr Harman could see chaotic situations arising on some contracting sites if grading were introduced. Most contracts used several grades of concrete, and the introduction of yet another variable was bound to lead to an increased risk of error. The contractor might have difficulty in getting the particular brand laid down in the specification, if it were too rigid, and even if he got what he wanted he would need separate storage for different grades, otherwise confusion was inevitable. What if he ran out of the higher grade ? The ganger, being a conscientious fellow, would not hold up the job on that account, for the use of a few bags of the lower grade would cause him no heartburn.

In Table 4, Dr Lea had quoted good control as 12 per cent coefficient of variation, whereas apparently at the Building Research Station a coefficient of 13·5 per cent had been obtained. Mr Harman said that he would be glad to have more information which might explain those figures.

Mr E. E. Morgan, dealing only with the rate of hardening, outlined his requirements by describing first what kind—not type—of cement he wanted ; secondly, what cements had been supplied ; and thirdly, how he hoped to obtain what he wanted.

First, he wanted a cement which would have—within practical limits—pre-determined breaking strengths at, say, 7 days, 28 days, and 3 months. For a building, minimum cube-crushing-strengths at 28 days were often specified, but the structure might not be required to carry its full working load until some fixed later period. Without a knowledge of the rate of hardening of the concrete it was not possible to produce a strictly economical design nor, by means of tests at early ages, to check whether or not the concrete would reach its specified ultimate strength at the later date.

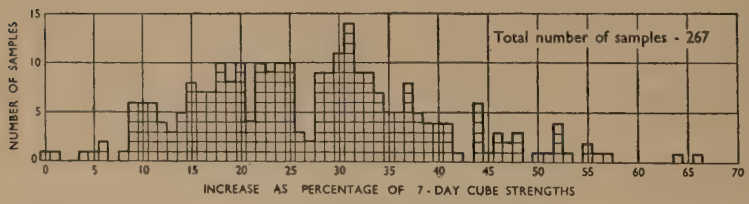
Mr Burke had said that there was little to chose between concretes made with various grades of cement after about a year. Mr Morgan accepted that, but practical requirements made it essential that he should be able to assess the probable strength at a much shorter period. For example, a concrete road constructed in short sections, with traffic passing, was a case where a knowledge of the rate of hardening, combined

with early strength tests, would have made it possible to open the road to traffic sooner than the recognized periods and thereby reduce the time to complete the work.

In general, when designing mixes, Mr Morgan would prefer to use a medium-strength cement with a reliable rate of hardening, rather than risk the uncertainty of an unknown cement, even though the latter might be expected to have a high ultimate strength.

Next, what cements were being supplied ? Mr Morgan showed a slide, reproduced in *Fig. 9*, which was prepared from an analysis of the test results of 267 samples of concrete made with cement supplied to the Heathrow works.[1] To eliminate a possible variation arising from the use of only one machine to test the 7-day cubes and another to test the 28-day cubes, only the strengths obtained when the same machine was used to test the cubes at both ages had been included in *Fig. 9*. That

Fig. 9



HEATHROW TEST RESULTS

(Increase in strength of 28-day cubes (average of 4) over strength of 7-day cubes (average of 2) as a percentage of 7-day strengths).

Figure depicted a histogram relative to the rates of hardening instead of the crushing strengths. For measuring the rates of hardening, Mr Morgan had used the 7-day strengths as his base and had expressed the 28-day strength as a percentage of the 7-day strength. For example, if the 7-day strength were 3,000 and the 28-day strength 4,000 lb. per square inch, there would have been an increase of 1,000 lb., or 33 per cent, over the 7-day strength. To make the histogram clearer to follow, Mr Morgan added by way of explanation that, of the 267 samples tested, there were nine having a rate of hardening of 33, fourteen of 31, and six of 10 per cent. *Fig. 9* showed that the rates for Heathrow had varied from 0 per cent to 65 per cent—a surprisingly large range. It might be thought that so high a range was exceptional. To indicate that such was not the case, Mr Morgan showed a slide depicting a histogram giving the rates of hardening of 158 samples of concrete made from the cement supplied to another runway works. In that case the lowest was 2 per cent and the highest 67 per cent. Recent tests carried out on 17 samples of concrete made with

[1] See ref. 2, p. 449.

ordinary cement (each sample consisting of two 7-day and three 28-day cubes), had produced rates of hardening ranging from 18 to 49 per cent.

Extending the analysis of the Heathrow test results, Mr Morgan had found that the lowest strength in the 33 per cent group of nine samples was 3,960 lb. per square inch and that the highest was 5,185, with an average strength of 4,850 lb. per square inch. In the 10 per cent group of six samples the lowest was 4,330, the highest 5,250, and the average was 4,800 lb. per square inch. Although the 28-day average strengths for both groups taken at random were almost the same, in the first case the concrete had reached only three-quarters of its 28-day strength within 7 days, whereas in the second case the comparative value was nine-tenths.

By testing for strength at 7 and 28 days, over a period of years, concrete made with cements of average quality, and using as before the 7-day strength as the base, it had been found that the average value for the rate of hardening for ordinary cement was about 50 per cent and that for rapid-hardening was about 30 per cent, but, with the wide variations in those rates which occurred in practice for single samples, it was clear that no reliance could be placed on those average values. Mr Morgan considered that, with the cement supplied at the present time, it was not possible to predict the 28-day strength from the 7-day-strength test result and the practice of using the 7-day strength as a guide was not recommended. He hoped, however, that it would be only a matter of time for the cement manufacturers to reduce those wide variations to within reasonably close limits.

Mr Burke had blamed the coal for the variations in hardening, and no doubt what he said was true, but it was an old problem. Mr Morgan had first met it in 1937, in the form of varying 28-day strengths, and Mr Sparkes had also raised the point in a Paper [1] published the following year. To some extent the causes of varying rates of hardening and strengths were inherent in raw materials and those materials differed from one works to another. Cement could not be used to its best advantage so far as its strength properties were concerned, because the manufacturers classed all their ordinary cements as the same and brought them all down to the lowest common denominator; similarly for rapid-hardening cement. Any one consignment might contain cement from more than one works, and that brought him to the third point—how he hoped to obtain what he wanted. He did not consider that there was any need to ask the manufacturers to alter their plant so as to produce a special low-strength cement—the proportions could be adjusted to cover that. Nor did he expect any spectacular changes in the material with large-scale mass-production in operation; it would not be economically possible nor sound practice at the present time. But he wanted them to follow Dr Murdock's recommendations (1) and (2) on p. 460, and label their products.

[1] F. N. Sparkes, "The Control of Concrete Quality." *Chem. and Ind.*, vol. 57, p. 1098 (19 Nov., 1938).

A clear indication should be included of rates of hardening, by giving within practical limits—not too wide—strengths at various ages, using a recognized method of test. Bag or bulk supplies should be so marked that they could be readily identified with the works where they were manufactured. Mr Morgan could then choose his source of supply, restricting his choice by practical considerations such as availability and haulage distance, and design his mixes to use the materials available to the best advantage.

Mr Morgan concluded by saying that he had, perhaps, made destructive rather than constructive criticism of the cement now being produced, but the fact that during the discussion Mr Snow had made a request for a low-grade cement—at a reduced price, it was true—for a low-quality concrete, and had complimented the industry on its chemists, had raised an interesting point. The manufacturers could say that their cements were so good that their customers were asking for something less good. They were entitled to a pat on the back and, when choosing their target, might have recalled the words of the poet, George Herbert :

" who aimeth at the sky
Shoots higher much than he that means a tree."

Paraphrasing his earlier remarks, Mr Morgan said that the sky occupied a a large area and could be reached by aiming in many directions. He was not particularly interested in what happened in the sky but would rather aim low and straight, following the arrow throughout its flight, and be sure of hitting the tree.

Dr A. R. Collins said he did not propose to speak directly on the main issue, but to deal with what he thought could be called some fundamental side issues. First, he would emphasize how important it was to dissociate the rate of hardening from the quality of cement. He showed a slide tabulating all the results he had available (and which could be strictly compared) of strength tests made at an early age and at 6 months. The tests were made on a range of different cements including four ordinary Portland, one rapid-hardening, one low heat, two Portland blast-furnace, and one sulphate-resisting. The Table showed that the 3-day strength of the individual cements ranged about the mean for all cements from — 38 per cent for the low-heat cement to + 66 per cent for the rapid-hardening cement, whilst at an age of 6 months there was only one cement which varied from the mean by more than 6 per cent.

That showed that even if different kinds of cement were used, there was a tendency for the strengths to become equal at about 6 months and that differences in strength shown at an early age were not representative of differences in quality but only of different rates of hardening.

A cement having a high early strength might or might not be as good as one with a much lower strength. Its resistance to chemical attack and to cracking might not be so good, and the rate of heat evolution might be

inconveniently high. The judging of cement quality by early strength had about the same justification as there would be in judging a man's ability to do a job by the number of marks he had obtained in one subject during an examination in his schooldays.

The cube strength of concrete at 7 or 28 days was a very convenient way of measuring its quality, so convenient in fact that other factors were liable to be forgotten. He urged engineers not to allow the means to obscure the end, and to consider very carefully whether a high early strength was really necessary before demanding it in a specification and not to ask for it merely because the test-cube results looked better. High early strength was usually only required to resist stresses during the construction of a building or to allow precast concrete to be handled. By suitable design or by the use of different construction techniques it might be possible to reduce the need for a high early strength with a possible increase in the ultimate factor of safety.

Dr Collins felt that strength grading of cement might well, in some circumstances, be a structural economy so far as materials were concerned, but it should be remembered that in order to grade one part of the cement output into a higher category it might be necessary to test practically all the cement. That would mean that the average cost of producing all cement would be increased, and the gain on the few structures requiring a high early strength might well be lost on the many for which it was not necessary.

Finally, he wished to refer to what Dr Faber had said about standards. There was one point which was often lost sight of in fixing standards. One did not make, test, and use cement like one made, tested, and used nuts and bolts. When nuts and bolts were made each one or each batch could be tested, and if it complied with the standard it could be sold. On the other hand, when cement was made it was tested by the maker and also by the user, but the results of the user's tests could not normally be available until the concrete had been placed. In addition, the testing methods were not so precise that small margins in quality could be tolerated. If it transpired that a cement was below standard after the concrete had been placed, a very difficult situation would arise. The cement maker would not therefore wish to risk that even a small proportion of his cement would not comply with the standard, and would therefore require a good working margin in strength. If the engineer wished to reduce the difference between the average strength and the minimum specified strength he had to be prepared to make his own tests before he used the cement.

Dr T. P. O'Sullivan said that the slide shown by Dr Collins had indicated four ordinary Portland cements, but he was still under the impression that in view of the large number of cements there were, even at a period of 6 months, probably greater variations than those shown. Assuming that those variations existed, it seemed that there should be some quality and price grading. Otherwise engineers were controlled by minimum

qualities; they would thus be in a similar position to that on building a job in steel where different qualities of steel were available, some being ordinary steel and some high tensile steel and they were to say "We do not quite know which are which, and so we shall put them all together and design on the basis of ordinary mild steel."

One aspect of the matter that struck one very forcibly was that if strength grading was accepted, there would be specifications which were more complicated than ever. What would be the position if there was a move in the direction of strength control rather than specifying the proportions of mixes? The demand for control in the quality of cement would surely become greater. He felt that, with certain qualifications such as specifying minimum quantities of cement and so forth, it would be possible to move in the direction of control by crushing strength, and strength grading would then become a matter of very great importance.

He was not very clear about what was the cause of the different strengths. Apart from the age of the plant, he felt that some manufacturers were in the happy position of having available materials with such a chemical nature that they lent themselves in production to a stronger cement than others, and that was an important point. Of course, if that grading system were to be accepted in conjunction with the specification of crushing strength, the manufacturers who had the best cements would have a demand for their products and those who were not in such a fortunate position would presumably find that if the supply exceeded the demand, the demand for the poorer quality cements would fall off. That brought in the whole question of commercial policy and price adjustment. The matter was obviously fraught with great difficulties, but he felt that it was not in the national interest to use a higher quality cement than was needed in a greater quantity than was needed. If higher quality cements were obtainable from certain plants, then it should be possible to reduce the quantity of cement used, so there could be a price adjustment.

Mr J. A. Derrington said that Dr Murdock's suggestion that the manufacturers of cement should publish more concise details of the characteristics of the cement which they produced would be welcomed by everyone. It was obviously more attractive to know something of the material that one was using, rather than merely to know the comparatively low level to which that material could fall before one could justifiably complain.

The variables which controlled the quality of the cement could be considered under two headings: one included those variables that arose inside any one particular cement works, such as the quality of the coal or the degree of supervision in the making of the cement, and the other covered those which arose between the products of different works which might be caused by the materials used in its manufacture or the manufacturing machinery.

The variations which arose under the first heading were perhaps more

acceptable because, for one thing, they had less effect on the concrete, and, secondly, the cement manufacturers stated that they were endeavouring to improve the situation. Those under the second heading were more serious, were less easily controlled, and were more costly to correct, although their disadvantages could be limited if the data on the product, for which Dr Murdock had asked, was given.

He could quote from recent experience on one contract where the coefficient of variation of 28-day cube results for one mix of concrete for which the cement was supplied by one works was 9·8 per cent, and that of another mix of concrete for which the cement was supplied from two different works was 14·1 per cent.

Considering the details given by Dr Lea, on the relation between design strength and coefficient of variation, either a greater degree of uniformity between the materials supplied by different works was desirable, or a better yardstick of measurement than a 28-day cube strength.

He felt, however, that before deciding on whether they needed another grade of cement, engineers would like to know more details on how the price would be built up—whether it would be purely on quality or whether it would depend to a large extent upon haulage distance. Obviously they did not want a situation to arise in which a limited quantity of high-quality cement was all absorbed by local builders, simply because it was cheaper for them to use it rather than to import cement of a poorer quality from a different works at a distance.

*** **Mr D. A. G. Reid** observed that one could have wished that the objections to the proposals set out in the Paper had been stated more fully. It was evident that the idea did not commend itself to the cement manufacturers, and it might be that there were serious objections from the contractor's point of view. In fact, those who had indicated opposition to the proposal had spoken mainly irrelevantly about quality and testing techniques.

Quality was a matter for specification and, if strength at 28 days was a specified requirement, it should not be ignored in favour of strength at 6 months or in favour of shrinkage after setting. One could sympathize with Dr Lea's uneasiness when considerations of early strength were emphasized but he (Dr Lea) was not opposed to dividing the existing range of normal Portland cements into two sections, and no one seemed to ask for more than that. Furthermore, even if high early strength became a first consideration, it was to be presumed that all concerned with the manufacture and use of cement would have their eyes open a little wider than had been the case 30 years ago when the search for high early strength had had such unfortunate consequences in respect of shrinkage effects.

On the question of testing technique, it was not out of place to recall the very determined attack on the problem more than 20 years ago. It

*** This and the following contribution were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

was the lack of sympathy of the manufacturers that prevented the full fruits of that work being gathered, and in particular, at that time they had displayed surprisingly little interest in the development of a test which was relatively insusceptible to the personal factor of testers in different laboratories. In fact there was a good reason to suppose that prediction of concrete strengths at different ages might be achieved by a test suitable for routine control of production so that, under normal conditions, the cement need not be in the structure before its properties could be assessed.

It was appreciated that strength grading would inevitably bring increased cost, and that was the real objection to the proposal which one would have liked to see more carefully examined. Only the manufacturers could reasonably assess what the increased cost might be, and it was a pity that they had made no serious attempt to do so and to make their contribution to the discussion.

Mr D. A. Stewart remarked that nowhere in the Paper was any indication given as to the variation in compressive strength between one batch and another which might be expected of cement derived from any one works. The general opinion among engineers was that that variation ranged between 15 and 20 per cent. It would be of great advantage to the structural designer if from any works the following information could be obtained :—

(1) Mean compressive strength at 7 and 28 days of the product over the previous 6 months.
(2) Coefficient of variation of the compressive strength of the cubes made in obtaining the data for (1) above, over the same period.

If grading was to be adopted, it should be on the price and not on the material. That might be done in the following way for ordinary Portland cement and rapid-hardening cement :—

(*a*) Obtain the national average compressive strength for the particular product when tested in a standard manner with standardized materials by an independent authority.
(*b*) Express the value of the product from each individual works as a percentage of the national strength.
(*c*) The charge for the individual works product would then be made on the basis of the percentage of national strength times the mean national price.

For example :—

Mean national price = 84*s*.

Works A $\frac{130}{100} \times 84s.$

Works B $\frac{87}{100} \times 84s.$

Most of the objections to using a cement of very good quality for making a low-quality concrete might be avoided by writing proper specifications for the concreting work. The specification should demand two properties only from the concrete, namely :

(*a*) minimum compressive strength at a particular period, and
(*b*) minimum density as cast in cube form at 12 hours.

The contractor should be left to obtain the required results, taking into consideration the aggregate available and the site and placing conditions.

Mr Snow, in reply, said that whilst it had been a very full and interesting discussion, no one had actually answered the question posed by him in the Introductory Note he had submitted. The nearest approach to any practical suggestion to deal with the question had emanated from Dr Faber, who had suggested a revision of the British Standard specification, which in his view was necessary in order to conserve cement and save cost.

Despite much of the scientific discussion, it was a fact that broadly speaking there were three classes of concrete work and therefore there would seem to be a need for three grades of cement. Mr Snow had been pleased to learn from Mr Burke's Note that there was a move on foot, by the cement manufacturers, to produce at least two grades of cement, which proved Mr Snow's point that there was a need for more than the one official grade, at one price, as at present operated.

From Mr Herman's remarks there seemed to be some doubt as to whether 18 : 1 concrete was largely used, but he could assure Mr Harman that as a backfiller, and for the protection of pipes in drainage trenches and work of a like nature, that mix was quite common. Obviously such concrete would not be resistant to sulphate attack, which required special consideration.

Dr Collins had stated that one should not relate quality to rate of hardening. In the present discussion, Mr Snow was not worried about that ; what concerned him was that, in particular contracts the type of cement, general specification, rate of hardening, compressive tests, etc., were laid down in the British Standard and referred to by the consulting engineer. The contractor endeavoured, using all his resources, to obtain a cement of the nature described and required, and yet a large proportion of the concrete had not stood up to the tests prescribed. The speed of the work was such that, had the specification been rigidly adhered to, large sections of the concrete would have had to be cut out, although, fortunately, in the majority of cases the concrete had hardened up over a period of time, but the consulting engineer and other professional advisors then had to consider whether to reject the concrete or allow it to remain. The practical difficulties were readily apparent.

Dr Faber had suggested that on such occasions many people got together, talked about the problem and then went home and left it alone. Mr Snow agreed that that was so.

Mr Snow suggested that if the concrete did not come up to the standard required by the specification and the manufacturers were not prepared to grade cement, the consulting engineer was entitled to say to his client that owing to the strength of the concrete being less than specified, the cost of the work should be proportionately reduced and such difference deducted from the cost of the cement. He felt that if such action was taken on a few occasions then the strength grading of cement would be rapidly accomplished.

Dr Lea, in reply, said that Dr Faber had apparently asked for the raising of the present minimum requirement of the British Standard to what Dr Lea had referred to as "Grade 2—cement for structural purposes." He had appreciated the argument, but there were types of jobs for which the weaker cement was more satisfactory in the long run, and he would view with some apprehension the disappearance of the existing minimum requirements for ordinary Portland cements.

In his introductory remarks, when referring to 12-per-cent coefficient of variation for good control, Dr Lea had quoted 13·5 per cent as the figure for the results of standard concrete tests at the Building Research Station, and Mr Harman had asked for more information on those figures. The standard tests, made over a period of years, included 119 samples of ordinary Portland cement and 71 samples of rapid-hardening Portland cement from a variety of works; and the coefficient of variation of 13·5 per cent therefore indicated the order of variation which could result from using cements from many works, even with laboratory control of the proportioning and mixing of the concrete.

The 12-per-cent coefficient of variation for good control, on the other hand, represented the order of variability which could be attained on a building site where the concreting operations were efficiently controlled and care was taken to minimize changes in the materials used. It was evident that one of the essentials of "good control" was a reasonable uniformity in the cement supplied throughout the work.

Dr Murdock, in reply, said that it seemed from Dr Collins's comments that ultimately there was only one grade of cement, and that any grading system would do no more than differentiate between the rates of hardening of various cements. If that were true, there was little point in having a grading system in so far as structural design was concerned, and its function became that of determining the time at which the shuttering could be removed and the time at which the structure could safely support its working load. It was very important that that difference in the rate of hardening of cements should be borne in mind when any specification or grading system was being considered. It seemed to Dr Murdock that the publication of more information by the cement companies on the strength and other properties of the cement they produced would be of more value now than a new grading system. He thought it was not possible to produce an improved grading system until considerably more knowledge

was available on the strength/age relationships of the various types and brands of cements now being produced.

The cost of transport had been mentioned. Knowing the cost of transport of building materials generally, he suggested that that would probably compel the use of local cements in most circumstances, with an adjustment in the mix proportions to produce the required strength, rather than the importation of a cement with higher early-strength characteristics from elsewhere.

Mr Burke, in reply, said that Dr Faber had clearly appreciated some of the various difficulties facing the cement industry. His contribution was essentially an argument in favour of higher strength requirements in the British Standard specification for cement. Inasmuch as his argument was based on the properties of currently produced cements, he was presumably advocating one higher grade, and not the two grades which had been suggested. The increase in specified strength could not, however, be more than about 20 per cent because the manufacturer had still to set his standard 30 per cent above specification to allow for variation between testing laboratories and other factors.

Dr Faber had attributed " enormous " discrepancies in concrete cube strengths principally to variations in cement. Variability in cement was admitted, but Dr Lea had been quoted as saying that variability arising from poor control would much outweigh that arising from the cement itself. How often was the control not as good as it should be ?

Mr Harman had given a refreshingly commonsense point of view on grading of cements. He had also instanced a case where a 12½-per-cent reduction in output was said to have resulted in better burning and satisfactory cement.

Manufacturing difficulties were likely to arise at any time but they were always resolved satisfactorily. The proof of that was that Dr Faber's case for a higher standard was based on the fact that very few cements failed to pass that standard—let alone B.S.12. One felt that the circumstances in the case quoted by Mr Harman must have been quite extraordinary.

In Mr Morgan's remarks the request for cement with less variability stood out very clearly. As stated in the Notes, that was the target at which the manufacturers were aiming. If it were attained, it should not be difficult to answer questions on rate of hardening and other properties of cement.

Dr O'Sullivan's comparison of cement and steel was not entirely a happy one. For one thing, steel was not normally mixed by contractors with other materials to form a composition whose strength was held to be a criterion of the steel's quality. Further, the only hardening of steel took place from the molten state before it was used, and under normal conditions there was little or no subsequent change in strength. In comparing cement with steel one should therefore consider the late or hardened,

rather than the early, strength of concrete. On that basis the variability of cement was not a serious matter.

Both Dr Murdock and Mr Derrington had asked for more information on the properties of cement other than strength. It was not practicable at present to give that in general terms but the manufacturers would do so in specific cases if asked.

Mr Reid had suggested that the cement manufacturers had made no serious attempt to calculate the effects on costs of the proposed grading.

The considerable experience of the cement manufacturers made it quite plain to them that the proposed grading of cement would increase costs. Some reasons had been briefly mentioned in the Notes and Dr Collins had mentioned one or two more. That was a qualitative assessment which Mr Reid appeared to accept. It must be appreciated that the proposals would mean considerable reorganization at every stage of manufacture, testing, storage, and distribution. A detailed estimate of the effect of the necessary changes would be a considerable task. The cement manufacturers, who were all busy men, had not felt that the demand justified undertaking the task, and they were even less convinced of its necessity now that the present discussion had taken place.

In spite of the clearly expressed doubts of Dr Lea, who was a widely respected authority in such matters, quality had been interpreted by several contributors as meaning simply strength at an early age. Mr Stewart had gone so far as to estimate the cash value of the cement on that basis; though he did not (perhaps wisely) suggest at what age the calculations should be made. His ratios would, of course, vary with age and would eventually tend towards unity.

There remained two points which Mr Stewart had raised and with which the cement industry would heartily agree. Specifications for concrete should require strength and density at stated times. It would perhaps be better if they could relate to the structure rather than to possibly unrepresentative cubes. The second point was that the aggregates available as well as site and placing conditions affected the concrete produced. Those factors and others, including workmanship, could (and did) quite easily lead to more trouble than variability in cement.

It was appreciated that under present circumstances a designer must have some basis to work on and a contractor must be able to fulfil a contract specification. The cement manufacturers felt that reduced variability would help in that direction. At the same time they could fairly ask, " How often was it really necessary for the specified strength to be reached at say 7 or 28 days " ? Dr Collins had dealt admirably with that point.

It was important that those sponsoring the proposals in the Notes should realize that they were asking for far-reaching changes on the basis of quite arbitrary design requirements, which were on most jobs probably unnecessarily stringent. Those requirements needed searching scrutiny.

It had often been said that when a cement with a greater rate of hardening was used, less cement could be used in the mix with resultant economy. Probably no one would deny that at any given water/cement ratio the final strength of the concrete depended upon the amount of cement per unit volume. If less cement were used, although the required 7 or 28 day strength were attained, the final strength would be less than that of a mix with more cement. That meant that the late factor of safety would have been reduced. The attention of designers should be drawn to that since it was a matter that did not appear to be fully appreciated.

One might conclude by asking a question. Would not the grading of design criteria according to actual need and the closer control of construction be a useful contribution to structural economy ? Perhaps that would be ungracious. The cement industry admitted that there was variability in cement and they intended to reduce it ; that would help to achieve economy in mix design. Further than that they could not go at the present time, especially in view of the demand for increased output of cement.

WORKS CONSTRUCTION DIVISION MEETING

12 May, 1953

Mr David M. Watson, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Works Construction Paper No. 24

"The Construction of Tignes Dam and Malgovert Tunnel"

by

Jean Pelletier

SYNOPSIS

After an introduction describing briefly the permanent works—the dam, the tunnels, the two power-houses, and the prestressed steel pipes at Malgovert—the first part of the Paper deals with the construction of the Tignes dam itself, located in the French Alps at an elevation of 6,000 feet.

The 850,000 cubic yards of concrete required were poured in three working seasons, each of 150 days. The concrete specification called for large-size aggregates, which gave rise to difficulties; powerful vibrators had to be used for consolidation. A description is given of the aggregate quarry, the screening and crushing plant, and the concrete-batching plant.

One 10-ton and two 20-ton cableways were used for transporting the concrete for the dam.

The second part of the Paper describes the construction of the 9½-mile-long tunnel, built to carry the water to the Malgovert power plant. The methods used to drive the tunnel in the bad ground encountered in several places are recounted. Particular difficulty was experienced in driving through a formation of wet crushed quartzite, and the third section of the Paper deals with the special grouting operations that were required to make tunnelling possible. Conditions were such that it took more than a year to drive 200 yards in this ground.

Introduction

The Tignes dam and the Malgovert tunnel form part of the Haute-Isère Hydro-Electric Scheme (Figs 1, Plate 1). This scheme was built for Electricité de France, at a cost of £50,000,000, to develop the River Isère, a tributary of the Rhône. Both the dam, which is one of the main features of the scheme, and the tunnel are located near the source of the River Isère, which rises in the French Alps close to the Italian and Swiss borders. The site of the works is 400 miles from Paris. Work was started in 1946 and was planned to be finished on the 31st October, 1952. The dam was actually completed on the 25th October, 1952. (*Figs 2* and *3*, facing pp. 488, 489.)

The most upstream feature is the Tignes dam, which is a concrete arch structure with a maximum height of 595 feet above bedrock, a thickness of 140 feet at the base, a crest length of 980 feet, and a volume of 850,000 cubic yards of concrete. The lay-out is shown in Fig. 4, Plate 2.

The reservoir, of 191,000 acre-feet capacity behind the dam, is the first one to control the flow of the river, which is fed by glaciers. It will supply the two power-houses of the project. The Brévières power plant, which is $\frac{2}{3}$ mile downstream of the dam, was already in operation at the end of 1952, with a flow of 1,765 cusecs under a head ranging from 775 to 318 feet. The water is carried through a 12·6-foot-diameter steel-lined tunnel, 3,000 feet long, and a 10·5-foot-diameter penstock, 720 feet long, to operate three Francis vertical turbines, each of 44,000 horse-power. The annual output at Brévières will amount to 150 million kilowatt-hours.

Downstream of the Brévières plant, after a small diversion reservoir of 240,000,000 cubic yards capacity, a tunnel of 175 square feet cross-section, $9\frac{1}{4}$ miles long, carries the water required for the Malgovert power plant. The tunnel ends in a surge tank 164 feet high and 60 feet in diameter. From this tank, two $7\frac{1}{4}$-foot-diameter pipelines, 5,700 feet long, feed the Malgovert power-house under a gross head of 2,460 feet and at a flow of 1,695 cusecs, each pipeline being dimensioned for a production of 203,000 horse-power. (See Figs 1, Plate 1.)

The two pipelines were made by the firm of Bouchayer and Viallet, using a special process called "*auto-frettage*." This self-hooping operation is performed cold : the hoops of superior-quality steel are placed on tubes which are expanded by internal hydraulic pressure until the hoops are stretched tight. The hoops may be of rolled steel straps or of cables. In this process, the metal of the pipe passes its elastic limit and acquires a permanent deformation. This type of pipe reduces the quantity of steel, thus saving time and money. At Malgovert, the pipelines required 4,500 tons of steel treated by this process, as compared with the 35,000 tons of ordinary steel that would otherwise have been necessary.

The steel used in the Malgovert pipelines had an ultimate strength of 79,600 lb. per square inch, whilst the strengths of the rolled steel hoops and the cables were 156,400 and 283,000 lb. per square inch respectively.

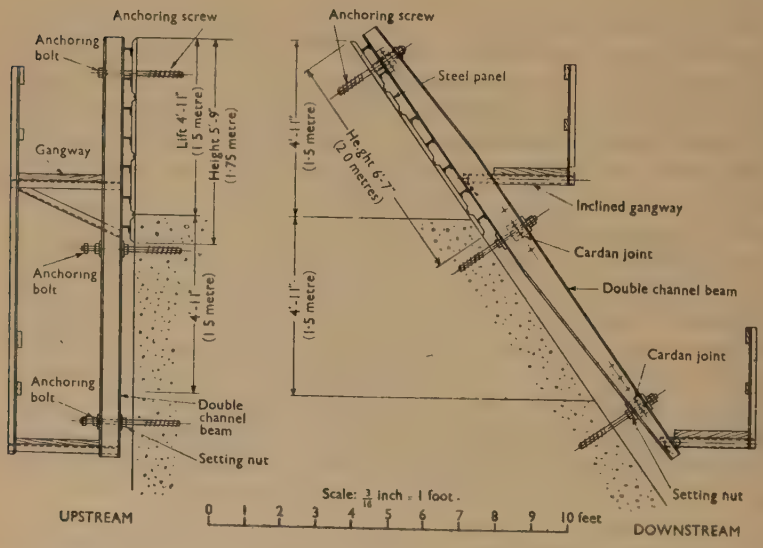
The Malgovert power plant has eight Pelton double-jet turbines driving four horizontal generators of 80,000 kilowatt-amperes, at 428 revolutions per minute, with an annual output of 550 million kilowatt-hours. This, the main power plant of the project, had begun to work in January 1953, with two generating sets in operation.

Irrespective of further flow-control on the River Isère downstream from Bourg-St-Maurice, where some large projects are to be carried out, the annual output of the Haute-Isère project will amount to 700 million kilowatt-hours.

Construction of the Dam

The main factor affecting the work was its location in the mountains. The high elevation (6,000 feet above sea level) allowed a working season of only 150 days each year. In order to pour 827,000 cubic yards of concrete in three working seasons, it was necessary to set up a highly mechanized plant, but the lay-out of the plant had been developed in 1946–47—just at the end of the war—and it was difficult to find all the necessary machines in Europe. For this reason, the main part of the equipment, including shovels, bulldozers, dumpers, cableways, and batching plant, came from the United States. On the other hand, the cement ropeway, crushing

Figs. 5



Shuttering for Dam

and storage plants, belt-conveyors, cars, radio sets, and steel forms were French (see *Figs 5* and Appendix 2). Finally, some dumpers and the four mixers of the concrete plant were supplied by English constructors.

Because of the short working seasons and the necessity to build the dam within a maximum of 3 years, three 8-hour shifts per day were worked. In 280 days of work, in 1951 and 1952, 714,300 cubic yards of concrete were placed—an average daily output of 2,550 cubic yards. The best days' outputs were 6,000 cubic yards (in 24 hours) on the 7th July, 1952, and 5,480 cubic yards on the 26th May, 1952, using the two 20-ton

cableways; the best month was August 1951, when 83,000 cubic yards of concrete was placed in 26 working days (*Fig. 6* and Appendix 1).

To keep such a plant working at maximum efficiency, special crews of highly skilled workmen were employed to carry out heavy maintenance on Sundays and light maintenance during the other days of the week.

Concrete for the Dam

The specification for the mass concrete of the Tignes dam was as follows:—

Sand up to $\frac{1}{8}$ inch	17 per cent by weight
Coarse aggregates, $\frac{1}{2}$ inch–2 inch	33 „ „
Boulders, 5 inch–10 inch	50 „ „
Cement, per cubic yard	378 lb.
Water/cement ratio	0·59
Entrained air	3·5 per cent by volume

The concrete with 10-inch aggregate was used for 80 per cent of the total mass. The remaining 20 per cent was concrete with a maximum aggregate size of 5 inches, which was used for the face of the dam. The two concretes were gap-graded. The gradings worked well because it was possible to use the whole of the normal output of the crushing plants.

The concrete gave good results after placing. For instance, 12-by-24-inch cylinders gave a compressive strength of about 2,400 lb. per square inch at 7 days.

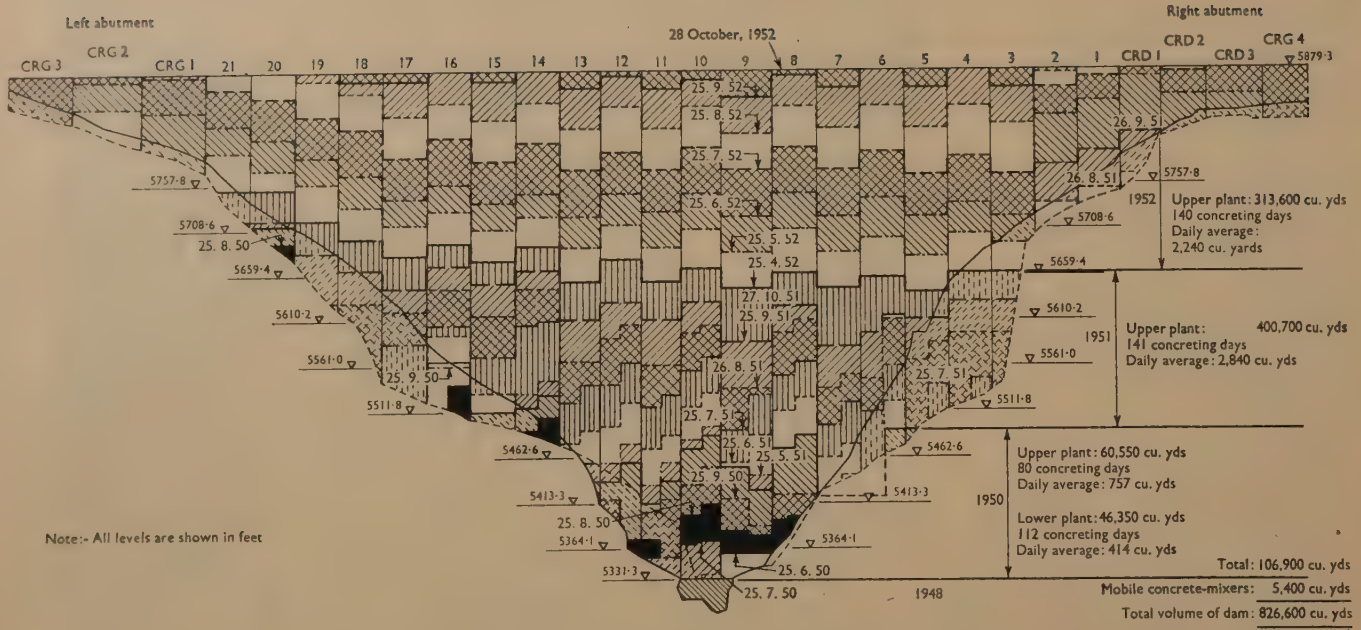
Difficulty was experienced in handling and placing the concrete, even with heavy vibrators (of the two-man electrical immersion type). The quality of the crushed sand was also found to be a cause of trouble. In fact, when the rock of the quarry was blasted and crushed, a large amount of dust was produced. This dust was removed by means of several types of cleaners, either in the secondary crushing plant or at the entrance to the stockpile. But when the rock was wet, as often happened in that locality, the cleaners did not work accurately and the amount of dust deposited in the sand stockpile comprised 20 per cent of the total weight. When this dust got into concrete, the air-entraining agent did not act and the concrete became stiff and unworkable and required more water; since the water/cement ratio had to be kept constant to comply with the specification, more cement had to be added.

Aggregate Quarry

The site of the rock quarry was in the forest of La Laye, 2 miles upstream of the dam, chosen because of the calcareous formation underlying it. To match a rate of concreting of more than 2,500 cubic yards per day, the aggregate had to be extracted with heavy equipment.

Two R.B. 29 T. well drills mounted on tracks were used to bore 10-inch-diameter holes for the primary blasting. These drills worked for 24 hours a day in three shifts. The adopted method of blasting was to use more

Fig. 6



CONCRETE PLACING PROGRESS, TIGNES DAM
(See also Appendix 1)

explosives in primary blasts in order to reduce the secondary blasting, thus saving time and money. In the hard limestone encountered, 1 lb. of dynamite, containing 67 per cent of nitro-glycerine, was used to the cubic metre. The rock fragmentation was good and, in the long run, this method proved to be economical. Ten or twelve holes were blasted at a time, twice a month ; that is to say, each blasting provided an average of 90,000 cubic yards of rock. The rock was loaded by two 3-cubic-yard diesel P. & H. shovels, and the hauling was done by eight 15-ton rear-dump Euclid trucks ; two trucks and one R.B. shovel were kept standing by.

The shovels and trucks worked only two shifts per day. During rush periods the two P. & H. shovels loaded an average of 450 trucks in two 8-hour shifts, and the R.B. shovel was also brought into use.

At the quarry, the rock was dumped into a 42-inch Allis-Chalmers primary gyratory crusher, which was able to break down the rock from maximum size to 10-inch size at a rate of 700 tons per hour. The aggregate was collected at the bottom of the crusher by a conveyor belt, which carried it to the silos of the twin-ropeway loading station. The two identical ropeways, each with a capacity of 200 tons per hour, carried the aggregate to the crushing and screening plant, 2 miles away.

Crushing and Screening Plant

The crushing and screening plant was composed of two identical and symmetrical lines of screens and crushers. The ungraded rocks were collected at the bottom of the unloading station by two conveyor belts, each feeding one of the crushing and screening lines.

The plant consisted of gyratory crushers, roller crushers, and vibrating screens, and produced five grades of aggregate :—

0– ⅛ inch ⅛– ½ „ ¼– 2 inches 2– 5 „ 5–10 „	[Approximate equivalents to metric sizes]

A large air-cleaning installation was provided inside the crushing plant, to improve working conditions.

Once classified, the aggregates were temporarily stored inside the plant in five bins, from which they were carried by two conveyor belts to the covered stockpiles, which provided for 50 hours of concreting. Two further conveyor belts, starting from a concrete tunnel built beneath the stockpiles, transported the aggregates to the top of the concrete plant.

This double line of machines greatly improved and stabilized the production.

Concrete Plant and Johnson Tower (*Figs 7* and *8*, facing p. 504)

One central concrete plant with a capacity of 235 cubic yards per hour was installed. This plant contained four 3-cubic-yard Koehring-Winget tilting-type concrete-mixers, made in England.

A covered storage hopper at the top of the plant was divided into five lateral compartments for the various grades of aggregate and a central compartment for the cement. A rotating chute discharged the aggregates from the belts into the proper silo. Aggregates, cement, and water were weigh-batched.

A central desk automatically controlled and registered all the batching operations. The controls for operating the gates of the bins, batchers, and the tilting of the mixers to discharge a batch were electro-pneumatic. The movement of the scale arms of the weighers used for measuring the batches controlled the closing of the bin gates when the correct weight had been reached.

After mixing, the concrete was discharged into two storage hoppers, each of 12 cubic yards capacity, from which the concrete was drawn off as required into 8-cubic-yard buckets travelling on flat cars hauled by diesel-electric tractors.

This concrete plant, made in the United States, was designed for using a 6-inch maximum aggregate size. During the winter of 1950–51, to meet the new specification drawn up by Electricité de France, it was necessary to modify the plant on the site in order to handle the 10-inch aggregate. The use of such large-size aggregate wore out the mixer-blades and heavy maintenance was necessary throughout the work.

Cableways (Fig. 4, Plate 2)

Three Lidgerwood electrically driven aerial cableways (one 10-ton and two 20-ton installations) were used on the construction of the dam. These were equipped for independent hoisting, traversing, and radial travel.

The main features of the cableways were as follows :—

Maximum load	20 tons	10 tons
Span	1,898 feet	1,944 feet
Height of head-towers	115 „	160 „
Lift of hook	from 660 „ below crest of dam to 65 feet above crest of dam	
Diameter of main suspension cable	3½ inches	2¼ inches
„ „ hoist rope	⅞ inch	
„ „ travelling rope	1 „	
„ „ button rope	⅞ „	
Distance between tracks of travelling head-towers	52 feet	73 feet
Length of head-tower front base	65 „	63 „

Approximate speeds and motor-powers :—

Lowering : 460 feet per min.	500 horse-power	250 horse-power
Hoisting : 300 „ „		
Traversing : 1,350 „ „		
Radial travelling : 100 „ „	two 75-horse-power motors	

The two 20-ton head-towers travelled on the same track and the 10-ton head-tower, which was taller than the others, ran on a separate track

behind them. This set-up permitted the 10-ton cableway to cross over the 20-ton cables. Limit switches prevented any collision between the 20-ton head-towers.

The fixed tail-tower was of latticed steel construction, and supported by three backstays anchored in large blocks of concrete set in a short tunnel driven into the mountain-side. The three main cables and the three button ropes were fitted to three swivel heads on the same tail-tower.

The travelling head-towers, which were of the four-legged type, were each balanced by a 350-ton concrete counterweight. The two front legs were mounted on ball-pivot bearings each carried on two six-wheeled bogies. Power for radial travelling was supplied by two electric motors, one on each rear leg.

The hoisting drum and the traversing drum were driven by the same motor, through a primary shaft which engaged with two large gear-wheels, one for each drum. Pneumatic clutches with wooden clutch-plates permitted running with both drums in gear (traversing) or with only one drum (hoisting or lowering), or running with the motor free. The drum clutches were controlled from the operator's cabin through two electromagnetic valves.

The load carriage of the 20-ton cableways ran on sixteen main cable-track wheels, mounted on compensating bogies; it contained pulleys for the hoisting and button ropes and was connected at either end to the traversing rope. When the carriage was going out from the head-tower, slack carriers supporting the hoisting rope were tripped out by the button rope and picked up by the horn on the return.

The cableways were controlled from the operator's cabin. Because the operators could not see the blocks of the dam they were guided by radio, and the connexion between operators and radio-men on the blocks worked so well that the buckets did not have to be moved by hand when lowered. The mechanical brakes were used only to hold the loads, six different speeds being provided by the motor, and a back-e.m.f.-type brake was used for slow down and stop.

The radial travelling speed was sufficiently fast to permit concreting on one monolith block with both cableways; one lowering its full bucket and coming in, while the other was hoisting its empty bucket and going out. This method saved a lot of time.

On Tignes dam, the cableways worked three 8-hour shifts, 6 days a week, heavy maintenance being carried out on Sundays. Time lost through mechanical breakdown was practically negligible.

Changing the cable was not considered as a breakdown repair, since it was always carried out on a Sunday, as part of the periodical maintenance. On the two 20-ton cableways, which were used for concreting, a new travelling rope was usually fitted after about 6 weeks of working 20 hours a day, 6 days a week. A new button rope had to be fitted after anything from a fortnight to 3 months of 20-hour-per-day working, and a new hoist rope

after 2 or 3 months. No particular wear was experienced on the main track cables.

The type of bucket used was an air-dumped 8-cubic-yard bucket of American (Gar-Bro) manufacture. For transporting excavation material and for hauling forms and equipment a heavy skip made of wood, carried by four-legged sling, was used.

Construction of the Malgovert Tunnel

As already stated, the construction of the Malgovert tunnel was begun in 1938. At that time, it formed part of a much smaller scheme called "La Gurraz Project". This tunnel was only 3 miles long and had a cross-sectional area of 75·3 square feet. The two generators of the power station supplied only 17,000 kilovolt-amperes each. During the German occupation, the scheme was re-designed to nearly its final size. It was decided to construct a large reservoir in the Tignes valley. The "La Gurraz" tunnel was lengthened as far as Longefoy, its section being enlarged to the present one. The scheme was taken over by Electricité de France in 1947, at the time of the nationalization of the electricity supply in France; the tunnel was further lengthened in order to feed the power plant on the Malgovert site at Bourg-St-Maurice.

Location and Access (*Figs 9*)

The 9½-mile-long tunnel was driven from twelve adits and from the downstream portal. The adits were actually numbered from 1 to 16, this being the number of adits previously planned, but some of them were omitted during the work, either because, with the efficient modern tunnelling methods used, their usefulness would not have justified their expense; or because they would have been located on or near a snow-slide site. The effective cross-sectional area of the tunnel is 160 square feet, whilst the excavation section ranges from 215 to 290 square feet according to the type of rock encountered. The elevation of the water-intake tunnel is 3,138 feet and of the terminal adit, 3,009 feet, the slope being 2·5 in 1,000 for the first 3 miles and then from 0·50 to 1·66 in 1,000. These various slopes were necessitated by unstable ground, which called for changes in the theoretical lay-out.

Since the tunnel was located on the left bank of the River Isère and the highway from Geneva to Nice was on the right bank, material had to be transported across the river by means specially designed for this project.

The canyon was very narrow along the first third of the tunnel length, so reversible ropeways were used, crossing the canyon in one span from the highway to the adits.

Farther downstream, the valley being too wide, it was not possible to erect that type of ropeway and it was necessary to use the existing climbing

Fig. 2

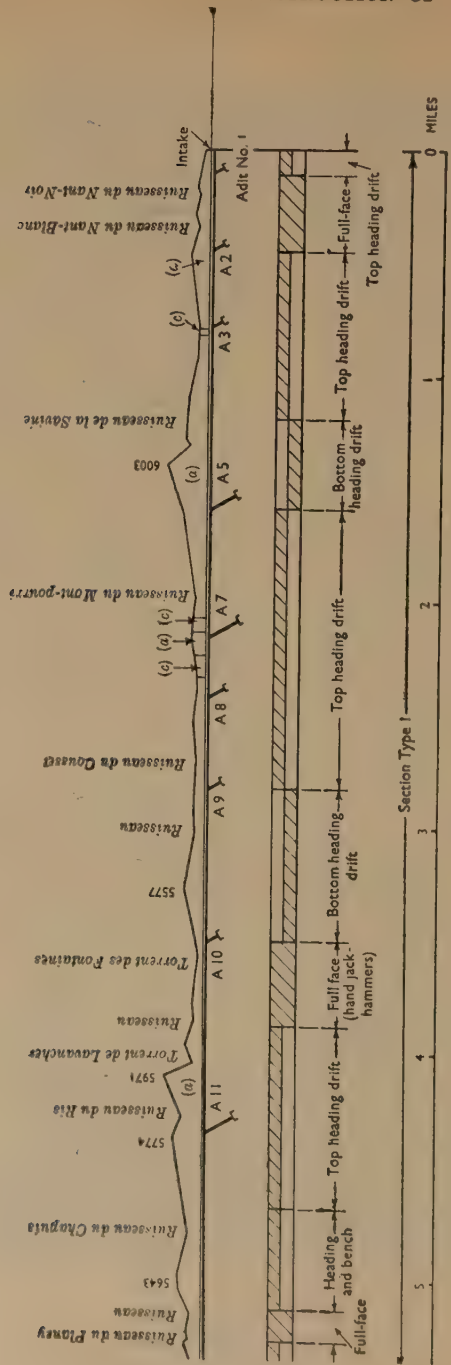
TIGNES DAM—AERIAL DOWNSTREAM VIEW
The travelling towers of the cableways can be seen on the left.

Fig. 3

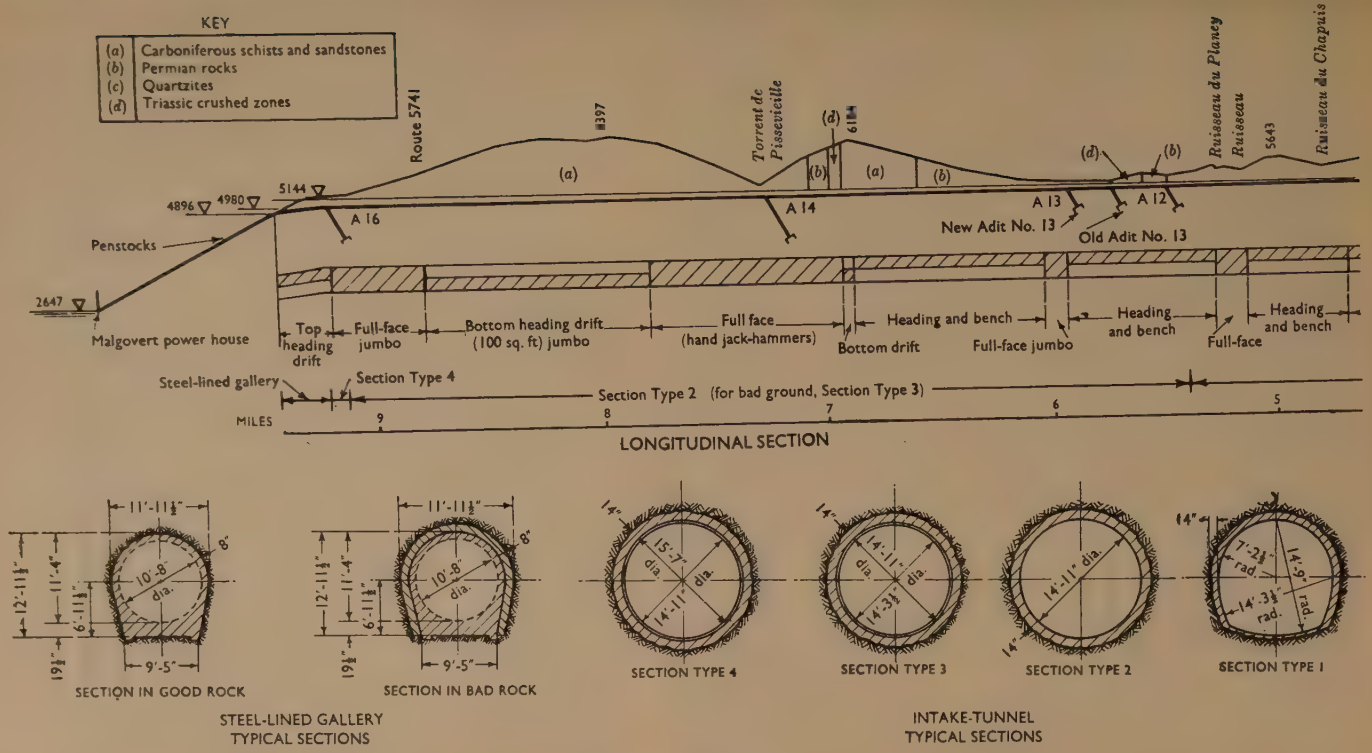
Figs 9 (a)

PLAN OF INTAKE TUNNEL

MALGOVERT TUNNEL


Intake
Adit No. 1
Ruisseau du Nant-Noir
Ruisseau du Nant-Blanc
Ruisseau de la Savine
Ruisseau du Mont-pourri
Ruisseau du Quard
Ruisseau
Torrent des Fontaines
Ruisseau
Torrent de Lavanchet
Ruisseau du Rio
Ruisseau de Chaputa
Ruisseau du Plomey
Ruisseau
A2
A3
A5
A7
A8
A9
A10
A11
(c)
(a)
6003
5577
5971
5774
5643
Top heading drift
Full-face
Bottom heading drift
Heading and bench
Full face (hand jack hammers)
Bottom heading drift
Section Type I
MILES
0
1
2
3
4
5

Figs 9 (b)


KEY
(a) Carboniferous schists and sandstones
(b) Permian rocks
(c) Quartzites
(d) Triassic crushed zones
Route 5741
8397
Torrent de Pisseviéille
6154
Ruisseau du Planey
Ruisseau
5643
Ruisseau du Chapuis
4896
4980
5144
2647
Penstocks
A 16
A 14
A 13
A 12
New Adit No. 13
Old Adit No. 13
Malgovert power house
Top heading drift
Full-face jumbo
Bottom heading drift (100 sq. ft) jumbo
Full face (hand jack-hammers)
Bottom drift
Heading and bench
Full-face jumbo
Heading and bench
Full-face
Heading and bench
Steel-lined gallery
Section Type 4
Section Type 2 (for bad ground, Section Type 3)
MILES
LONGITUDINAL SECTION
11'-11½"
12'-11½"
11'-4"
6'-11½"
19½"
8"
10'-8" dia.
9'-5"
SECTION IN GOOD ROCK
SECTION IN BAD ROCK
STEEL-LINED GALLERY TYPICAL SECTIONS
14"
15'-7" dia.
14'-11" dia.
SECTION TYPE 4
14'-11" dia.
14'-3½" dia.
SECTION TYPE 3
14'-11" dia.
SECTION TYPE 2
7'-2½" rad.
14'-9" rad.
14'-3½" rad.
SECTION TYPE 1
INTAKE-TUNNEL TYPICAL SECTIONS

paths after improving and sometimes lengthening them for some miles. As a precaution, in case of severe snow-falls, reversible ropeways were mounted on wooden frames up the mountain slope. During the six winters through which the work lasted, however, heavy tracked snow-ploughs kept the roads open with complete success and traffic stoppages never exceeded 2 days at a time. During the worst winter, a total fall of 28 feet of snow was recorded.

According to the previous geological surveys, the tunnel would have to be driven in primary ground ; the first part, along the narrow valley, in a metamorphic schist of a carboniferous nature, the second part in the micaceous schist of the permian ground. Certain geologists were concerned about the triassic Alpine formation, gypsum quartzite, already encountered in the next parallel valley beyond the mountain where the tunnel was to be driven. These Alpine triassics were found each time, revealing a *charriage*, that is, a big fault, the triassic having acted as a lubricant when the fault occurred. This had brought about the crushing of the triassic rocks. This formation was, in fact, encountered at adit No. 13, and 2 years were spent in driving 524 feet of tunnel through it fortunately without any casualties.

Downstream of the triassic formation, the tunnel was in the schist from the permian formation, but unfortunately, after a few hundred yards, a rather soft carboniferous schist was encountered, with more narrow coal seams. All these strata were of broken rocks, badly faulted, carrying water and needing heavy supports.

Plant and Site Installation

The installations provided in 1938–40 for the old " La Gurraz " tunnel were quite inadequate and, after the war, had to be changed in order to cater for the new project. For example, a camp had to be built for 250 men at adit No. 5 ; at the site, an aggregate quarry was opened, and a crushing, washing, and screening plant was erected. At adit No. 9, a reversible ropeway was modified for the use of the men, and the one used at adit No. 11 was improved to carry loads up to 2 tons. Two-stage modern compressors were substituted for the old ones at adits Nos 1, 2, 5, 9, and 11, comprising a total of 1,000 horse-power.

In the downstream part, it was originally planned to use only two adits (Nos 13 and 16) with 5,800 yards between them for the drilling operations ; the reason being that the location of adit No. 15 would have been difficult of access, whilst adit No. 14 was near a snow-slide site. Soon after encountering difficulties at adit No. 13, however, a small adit was driven at No. 14 in order to reconnoitre the ground. This was a carboniferous shale, in which it was much easier to drive than in those encountered at the two other adits. Later, when it was established that the rate of driving from adits Nos 13 and 16 would necessarily be very slow, it

became imperative that adit No. 14 should be used for drilling operations in order to save time.

A large reversible ropeway had to be built, capable of carrying the men or 5-ton loads, and extensive temporary works had to be constructed to provide full protection against snow-slides.

Two accidents were caused by snow in 1952. On the first occasion, a snow-slide demolished the local field offices of the contractor and Electricité de France, fortunately without any casualties. The second accident was caused by a snow-laden tree falling down a cliff on to a balanced ropeway and jerking one of the cars off the rope. The other car, with eight passengers, started to descend rapidly, but fortunately came to a stop hanging about 80 yards above the ground, without harm to the passengers.

Equipment used for Drilling and Lining Operations

Since work had begun on a small scale before the war in the upstream section of the tunnel, a 2-foot-gauge track and 17½-cubic-foot tilting wagons were available for this section, but when work was started on the lengths between adits Nos 12 and 16, it was decided to use 1-metre-gauge track and wagons for this section.

At first, the wagons were hauled by donkeys. This primitive means of haulage was later replaced by small French 16-horse-power electric battery locomotives. After 1947, the side-tipping wagons were changed for American 2-cubic-yard wagons, which ran on the 2-foot-gauge track, and were hauled by new battery locomotives of 25 horse-power.

In the downstream part—that is, between adits Nos 12 and 16—metre-gauge rolling stock, equipped with automatic couplings, was used. This consisted of American 5-cubic-yard wagons, designed for hand dumping on both sides, hauled by French 30-horse-power battery locomotives, which could be used in two multiple units with only one driver. These locomotives, designed by the Author, can be converted for use on either 2-foot or 1-metre gauge, merely by changing the axles.

In 1950 it became necessary to supplement the equipment by thirty new wagons of the same type, but of French manufacture, with French diesel locomotives of 70 horse-power. These diesel locomotives were first used only in open air to marshal the wagons in the adit yard. After a while, however, the good ventilation in the galleries made it possible to use them to haul the aggregate train from the feeding hoppers to the lining plants, and steam locomotives were used for the marshalling. The aggregates and the cement were transported in special single-batch containers. The 5-cubic-yard wagons, after removal of the spoil, were converted for carrying three containers. Diesel locomotives could haul trains of eleven wagons, the gross weight being 70 metric tons. In addition to these converted wagons, old wagons carrying only two containers were used at adit No. 16.

Drilling Operations

Drilling methods were modified in the course of the work, according to the different strata encountered.

Upstream section between intake and adit No. 12.—Three different methods were used :—

(1) 65-square-foot pilot tunnel in invert.
(2) 65-square-foot pilot tunnel in crown.
(3) Full face (in very few places).

The ground behaviour did not give rise to any difficulty, except for the last 1,000 feet, where heavy steel supports were needed. The headings were driven by means of hand jack-hammers, using hexagonal steel rods and forged bits of 1-inch diameter.

The loading units consisted of Eimco type 21 shovels, and the hauling units were 1- to 1½-cubic-yard wagons.

Downstream section between adits Nos 13 and 16.—When the main tunnelling operations in this section were started, in January 1948, it had already been decided to try the full-face method suggested by the geologists.

Two large jumbos were available, carrying eight DA. 35 Ingersoll-Rand drifters working with Timken bits. These jumbos could be used as "cherry pickers" (see *Figs 10*) during the mucking operations, which were done by an American Conway type 75 shovel. Forty-eight holes were needed to drive 10 feet at a time, in a 290-square-foot cross-section.

As the work proceeded, polygonal steel supports were erected at 4-foot centres along the heading after each blast.

In May 1949, the quality of the ground was steadily deteriorating, and it became necessary to revert to the so-called "Belgian" method, that is, using a top heading and successive enlargements. With this method, only hand jack-hammers were used with ordinary rods and either forged bits or jack bits. The 2-foot-gauge equipment had to be used in the top heading. As soon as better ground was encountered the heading-and-bench method was resumed, and the two methods were thus used alternately between adits Nos 13 and 14, until the break-through. The bench was then excavated, working simultaneously from adits Nos 13 and 14, and using also the ordinary jack-hammers. The same system was used between adits Nos 11 and 12, except that a British-made Conway type 60 electric shovel was used instead of the type 75.

Between adits Nos 14 and 16, a different method was adopted, which previous experience had indicated to be quicker. It had often been found that, with a smaller gallery, support was not necessary immediately after blasting, the spoil being removed and driving being generally resumed before support became essential. It was decided to drive a 100-square-foot invert gallery, using drifters for full-face operation. This seemed to give the best results in that particular ground. Two small jumbos with four

DA. 35 drifters were used, one with the boom equipment and long rods, the other equipped with ordinary rods. Ordinary jack-bits were used in shales and schist, and carbide jack-bits in primary sandstone ground. In the latter strata, found unfortunately only for a length of 700 feet, supports were not necessary at all. Mucking operations were carried out with an Eimco type 40 pneumatic shovel, which was found to give the best results. This tunnel was started in April 1951, and the break-through was made in January 1952.

The widening of the small pilot gallery was carried out either in one operation or in a number of steps, depending upon the type of ground. The work of widening the pilot gallery was started in a number of places simultaneously, so that the tunnel could be completed to schedule. *Figs 11* show the different polygonal steel supports and the arch ribs erected on a bench at either side. A longitudinal concrete sill was laid along each bench when it was found necessary to tie the arches together.

With these methods it took 9 months to drive the 1,531 yards between the headings from adits Nos 14 and 16, the latter having been driven for only 8 months. The enlarging operations of the same section took 6 months, the lining operation being carried out at the same time. In all, 17 months were necessary to complete this part of the tunnel.

Difficulties Encountered during the Work

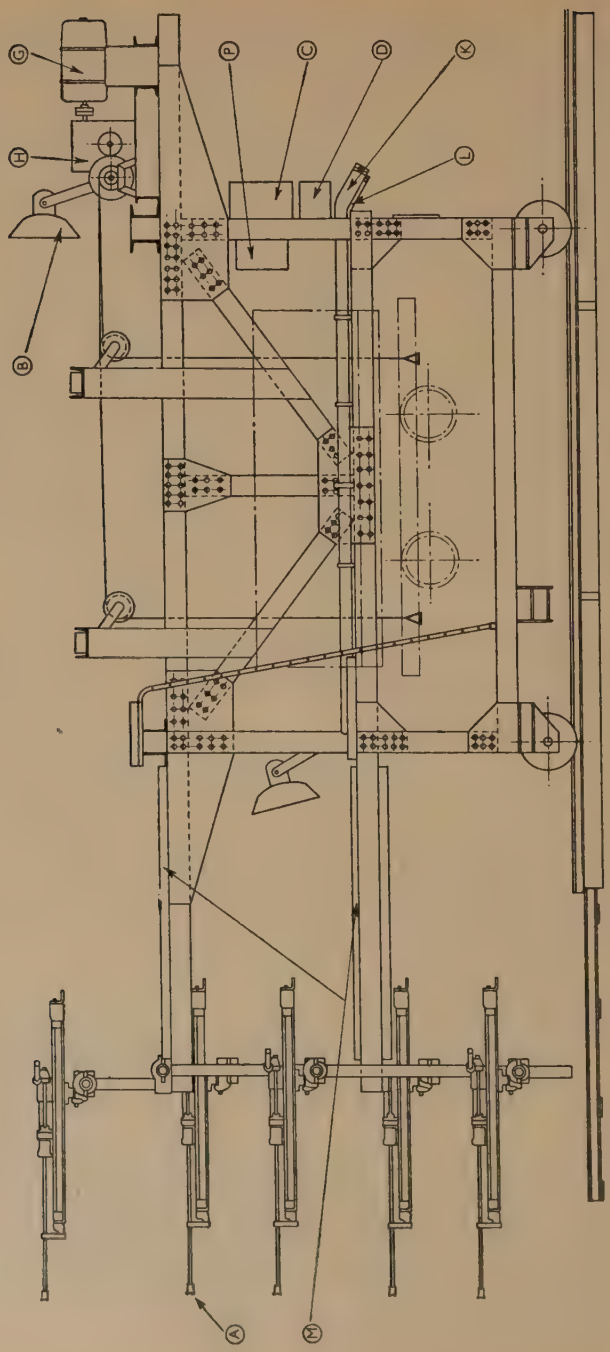
Difficulty arose as soon as work started from adit No. 13 : the first setback was a caving-in of the tunnel, with inflow of sand and water, and collapse of the overburden.

Downstream of adit No. 13, the ground formations of a permian-carboniferous and then of a carboniferous nature gave rise to much trouble. Several workmen and an engineer were killed by a failure of the rock. The ground pressure was so great that steel ribs to support the tunnel had to be spaced closer and closer together as work proceeded.

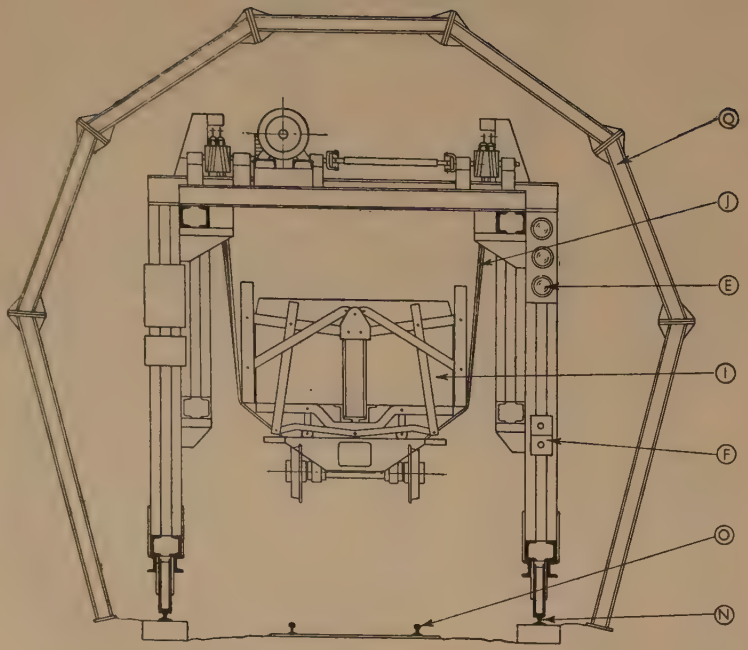
After a mile of driving with the full-face method, the top-heading-and-bench method had to be adopted, and later only the top heading. At that time, it was decided to start tunnelling from adit No. 14 with two full-face headings, upstream and downstream, since the carboniferous ground encountered there was thought to be of better quality.

In the meantime, the heading from adit No. 16 showed signs of trouble. The schist formation was so bad that heavy steel ribs, at 1-foot centres, were overloaded and deformed, and very often had to be replaced after only a few days.

At 150 feet from P10 (see *Figs 9* (*a*)), the tunnel ran into swelling ground —a stratum of badly crushed shales located near a fault. These shales were fairly dry and resembled a stiff clay in consistency, which made driving easy. But a few days after excavating, this ground became wet, swelling and developing such a heavy pressure that the steel supports failed, even if set flange-to-flange.



Figs 10

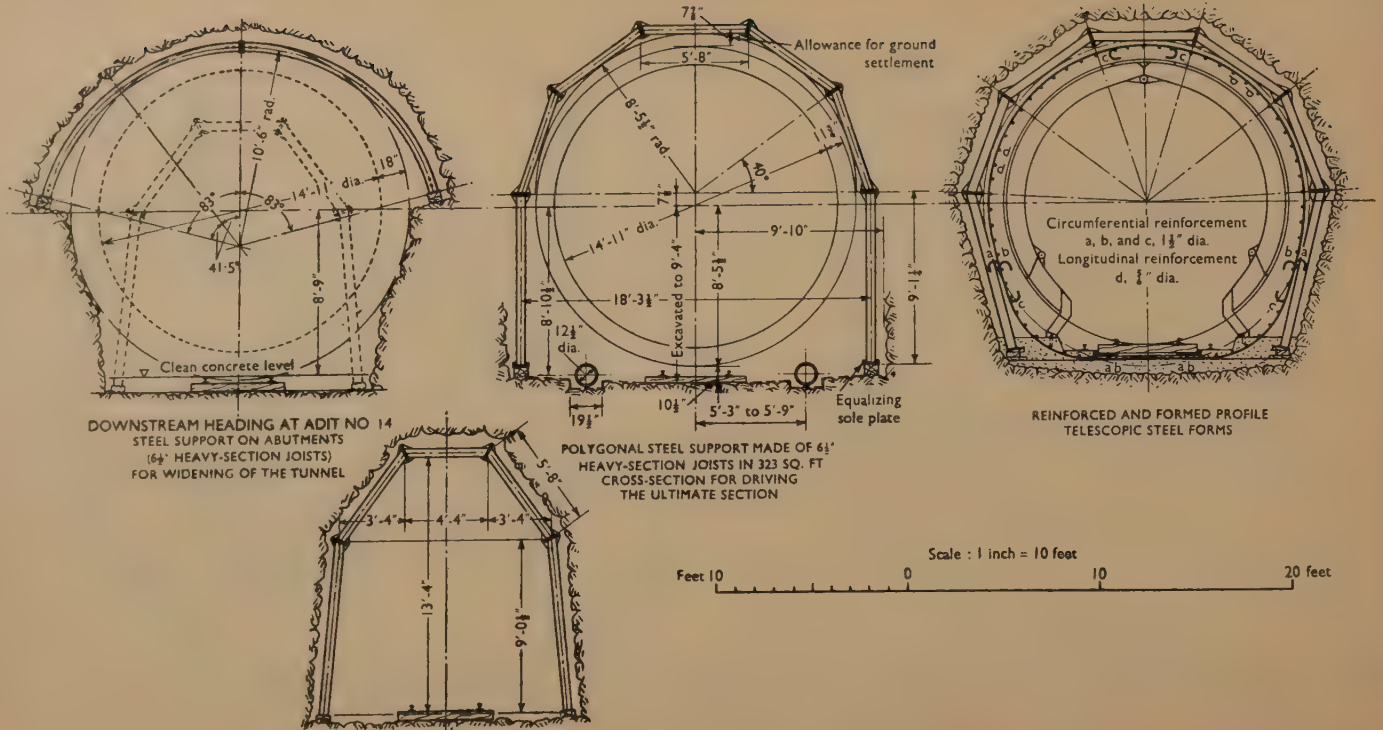


KEY

A—9 drifters DA. 35
B—110-volt light
C—380/110-volt transformer
D—110/24-volt transformer
E—signal lights
F—push-botton control (" up " and " down ")
G—380-volt motor for cherry-picker
H—differential
I—Western wagon half raised
J—hoisting cable
K—4" compressed-air pipe
L—2½" grouting-water pipe
M—working platform
N—jumbo siding rails on wood blocks
O—1-metre-gauge track on steel ties
P—cut-out for cherry-picker motor
Q—steel support

Drill Carriage

Figs 11



DOWNSTREAM HEADING AT ADIT NO 14
STEEL SUPPORT ON ABUTMENTS
(6½" HEAVY-SECTION JOISTS)
FOR WIDENING OF THE TUNNEL

POLYGONAL STEEL SUPPORT MADE OF 6½"
HEAVY-SECTION JOISTS IN 323 SQ. FT
CROSS-SECTION FOR DRIVING
THE ULTIMATE SECTION

REINFORCED AND FORMED PROFILE
TELESCOPIC STEEL FORMS

DOWNSTREAM HEADING AT ADIT NO. 14
STEEL SUPPORT IN BASE SECTION OF 129 SQ. FT
(6½" HEAVY-SECTION JOISTS)

It once happened that the track rose up to 4 feet in a few hours, while at the same time, all the ribs of the section failed. Orders were given to stop this heading and to try a diversion tunnel. A new tunnel was therefore started, 394 feet back, and driven toward the mountain side (points P9 and P8 on *Figs 9 (a)*). Finally, good carboniferous schists allowed driving to continue at an increased rate in this heading, until it was slowed down once again by large inflows of water from faults.

As result of a failure of the roof near a fault, a heavy inflow of water (nearly 4,000 gallons per minute) flooded the tunnel. Yet again the work had to be stopped and several pumps had to be brought into action. Several months elapsed before the heading could be resumed.

Until the break-through from adit No. 16 was achieved, a large pumping plant had to be kept working almost continuously to counteract an inflow of up to 4,500 gallons per minute, in spite of the fact that this section was driven uphill.

At the same time, the downhill heading from adit No. 14 encountered soft coal seams of variable thickness, and a few days after excavation, a serious squeeze occurred. As previously mentioned, the pilot bottom-drift method was used. Contrary to what happened at adit No. 16, practically no water inflow occurred, but as a precaution against flooding a large battery of pumps was installed as near as possible to the face. Strange though it may seem, this was never used, although the last blasting in this heading was in almost wet ground.

In order to get back on schedule, the completion date being November 1952, it was necessary to treble the plant for the concrete lining. All tunnel work was, however, duly completed on the 22nd November, 1952.

Lining Operations (*Figs 11*, *Fig. 12*, facing p. 505, and Figs 13, Plate 2)

Since the Malgovert tunnel construction was behind schedule, the concrete lining was started while drilling was still in progress at the heading.

At adit No. 16, lining operations were commenced in 1950. At first, the plant used consisted of a concrete-pump fed by a mixer, the whole plant being mounted on a flat bogie wagon running either on the main metre-gauge track or on a specially built siding. Results were unsatisfactory because of the bad quality of the sand, which lacked fines. It became necessary to change to another sand and add fines into the mixers.

In 1951 the concrete lining was also started from adits Nos 13 and 14 with a similar plant. In the meantime, the concrete pumps had been replaced by air placers. The concrete production for one train working three shifts ranged from 100 to 130 cubic yards per day, the best output recorded being 210 cubic yards in three shifts at adit No. 16.

The mobile concrete plants of one or two mixers were fed with dry batches from an outside batching plant.

Two different methods were used for placing the concrete in the shutters. With the first method, the train was moved slowly along at the

end of a 980-foot length of pipe to keep pace with the concreting. With the second, the train was kept stationary and moved only at the end of each week, the pipeline being shortened as the concrete was placed.

Concrete

For reinforced concrete, 650 lb. of metallurgic cement to the cubic metre was used. The water/cement ratio was 0·64, the sand/aggregates ratio 0·45. Tests on samples at 7 days showed a compressive strength of 1,991 lb. per square inch.

For plain concrete, 550 lb. of cement was used to the cubic metre. The water/cement ratio was 0·60 and the sand/aggregates ratio 0·40. Tests on samples at 7 days showed a compressive strength of 1,706 lb. per square inch.

Collapsible steel forms were used for lining the roof and the walls, the keying operations being performed after the wall had been poured by pumping the concrete through the pipe towards the key crown. The forms were set in lengths of 35 feet and vibrators of 4 inches diameter were used.

It was originally intended to pour the invert as a final operation after removing the tracks. This was found to be impossible, owing to various other operations that had to be performed at the same time, and which required the use of the tracks. It was therefore necessary to lift the tracks and set them on transverse wooden beams in order to permit work under the ties. This operation might appear to be very complicated and expensive, but in fact, with a little experience, it was possible to carry out this work easily and quickly. The concrete was mixed in a plant in the gallery and transported to the site by wagons or pipes. The best length done in a day was 220 yards, the average per plant being only 140 yards.

Sinking the Surge Tank

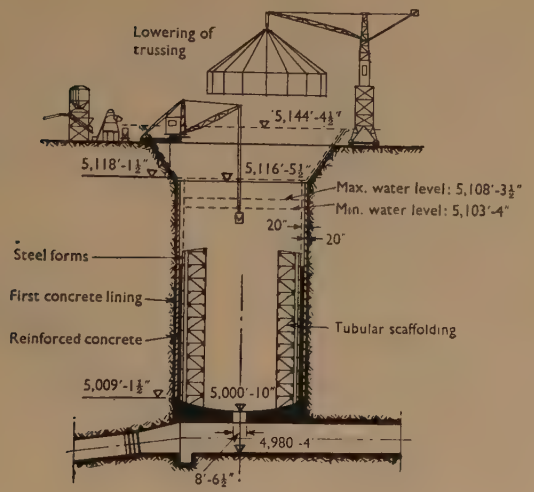
The surge tank is in the form of a 60-foot-diameter cylinder, 116 feet 6 inches deep, surmounted by a truncated cone of 100 feet base diameter, levelled off at about 10 feet above ground level. (*Figs 14.*)

Excavation

Excavation was started at the beginning of 1948, the work being carried out as follows.

A shaft, 6½ feet square, was first excavated from ground level, and then opened out in horizontal steps of 10 feet, working from top to bottom. The spoil fell into the existing shaft at the bottom of the tank, where it was recovered by wagons. Crews of 15 to 20 men worked three shifts per day, and the excavation was completed in April 1950.

Figs. 14 (a)



SECTION

SURGE SHAFT

Concrete Lining

The concrete lining consisted of :

(1) a concrete retaining wall ; and
(2) a final concrete lining.

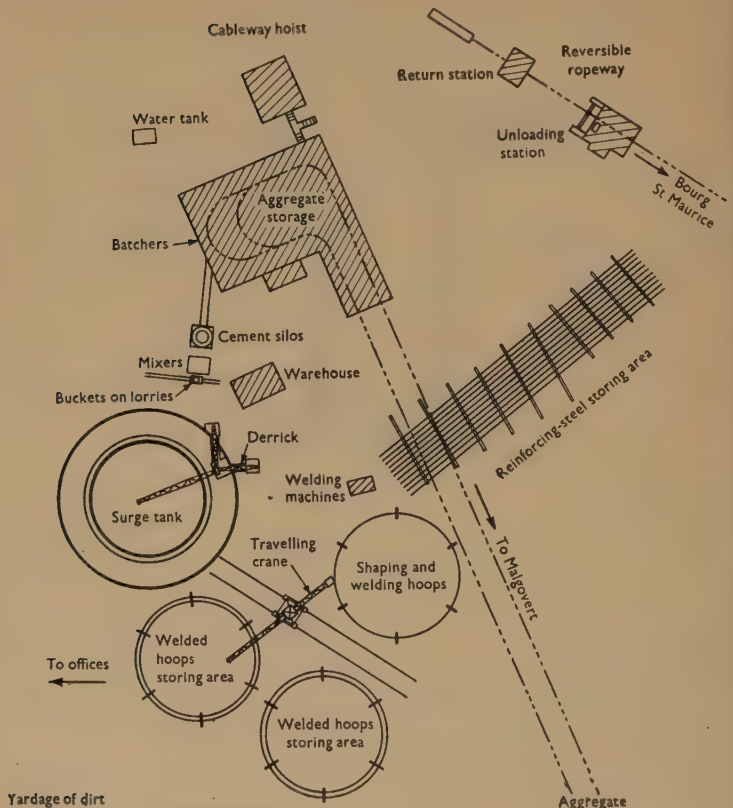
The retaining-wall concrete was poured immediately after the shaft had been excavated to its final diameter, so that placing progressed from top to bottom. The final concrete lining was poured after excavation had been fully completed.

Bulk cement and aggregates were transported by ropeway up to the silos of a batching plant erected close by the surge tank. The aggregates were batched by volume and carried to the mixers by a belt-conveyor, the cement being supplied by Fuller-Kinyon pumps.

The reinforcing steel, in the form of welded hoops, was lowered into the surge tank by a crane. When lowered into position, the hoops were welded to special vertical rods. Prefabricated expanded-metal blankets were set at the same time as the reinforcing steels.

The concrete was poured in 20-inch lifts. Crews of 25 men worked three shifts per day, and a crew of 10 men, working only a single shift in the day-time, erected the reinforcement. The work was finished on the 1st August, 1951.

Figs. 14 (b)



SITE LAY-OUT

SURGE SHAFT

TUNNELLING THROUGH HIGH-PRESSURE WET QUARTZITE
(See *Fig. 15* and Appendix 5)

In 1946, when driving adit No. 13, a wet crushed quartzite seam was encountered at about 426 feet from the portal. The same ground was also met 1,080 feet from the portal of another tunnel that was driven downstream in order to connect with the theoretical lay-out. Each time the heading collapsed, and the tunnel was filled with wet crushed white quartzite sand. After removing it by forepoling, test bores were drilled

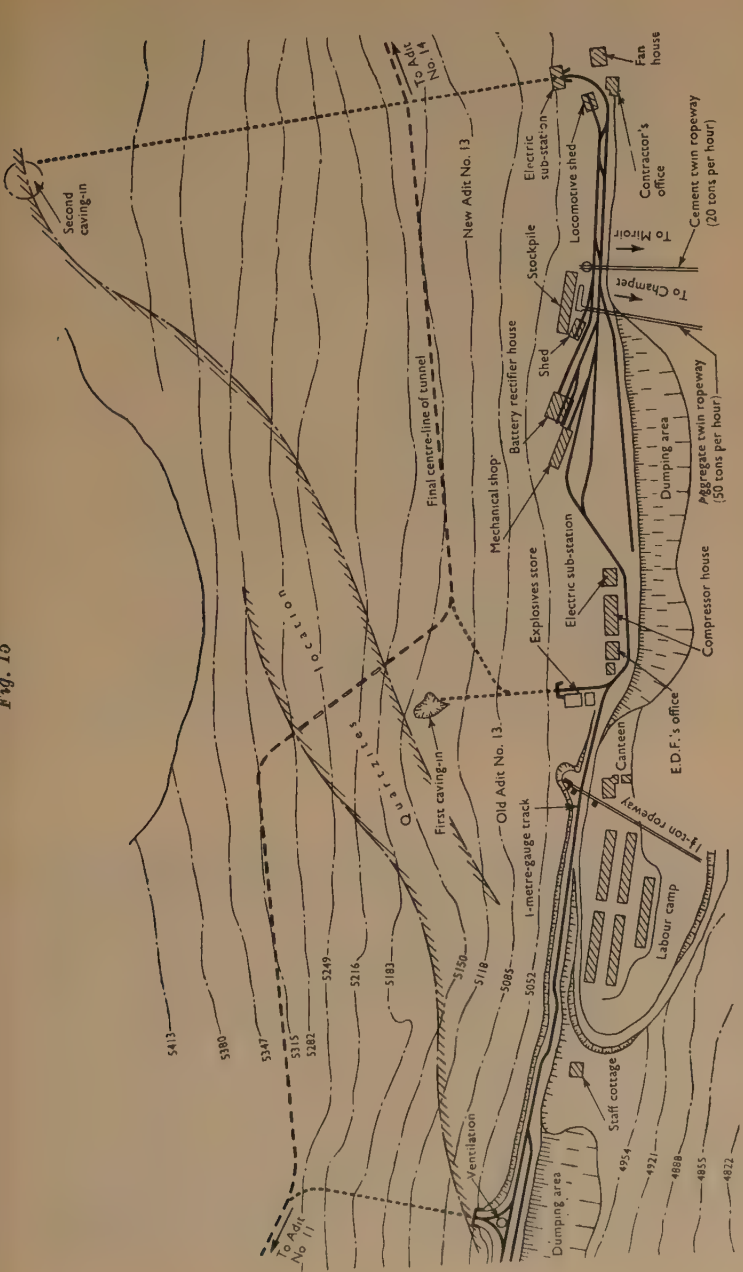


Fig. 15

PLANT LAY-OUT AT THE BADLY FAULTED ZONE NEAR ADIT No. 13

and the pressure of the water was found to be 115 lb. per square inch. Trial bores from the surface revealed the extent of the danger zone and also that the tunnel would have to cross it at least once.

The design lay-out from point C1 to C2 (*Figs 9*), 524 feet in length, was composed as follows :—

Comparatively good schist	106 feet length
Crushed schist	32 „
Fine crushed quartzite	65 „
Composite quartzite (boulders and sand)	109 „
Fine crushed quartzite	6·5 „
Trachytic tuff	12·7 „
Quartzite-phyllite sand	22·9 „
Crushed schist	62·2 „
Comparatively good schist	101 „

A specialized contractor was called in to carry out the grouting operations.

A gallery was first driven from the old adit No. 13 at point A to point C2 (see *Figs 9* and *15*). The cross-section of this gallery was 86 square feet. From point C2 the tunnel was driven full face towards C1 for a length of 49 feet. This length acted as a base from which the specialist contractor could start work.

From the first base, a cylinder of ground was consolidated by a series of horizontal bores. This cylinder was concentric to the tunnel and varied from 81·9 to 147 feet in length at different stages of the work. After the ground had been consolidated, excavation was carried out for only part of the consolidated length. This excavated length was used as a base for

TABLE 1

Downstream heading	*Upstream heading*
Stage No. 1.—Strengthening of the crushed schists of the wall.	
Stage No. 2.—Excavation up to 16·3 feet from the schist-quartzite area.	
Stage No. 3.—Strengthening of the crushed quartzites and of some composite quartzites.	
Stage No. 4.—Corresponding excavation.	
	Stage No. 5.—Strengthening of the schist in the crown.
	Stage No. 6.—Corresponding excavation.
Stage No. 7.—Strengthening of the rest of the composite quartzites.	
Stage No. 8.—Composite quartzites.	
Stage No. 9.—Strengthening of junction length by ground consolidation.	
Stage No. 10.—End of excavation and break-through.	
Stage No. 11.—Final concrete lining.	

Fig. 7

Batching and Mixing Plant, Tignes Dam

Fig. 8

Tignes Dam—Plateau des Boisses

Photograph shows covered stockpiles, towers, secondary crushing plant, and batching and mixing plant.

Fig. 12

Adit No. 16. Shuttering and Dolly

Fig. 17

Adit No. 13. Heading and Bench in Grouted Quartzites

a further length of boring for consolidation. This method of alternate driving and consolidation was used for the 524-foot length through the water-bearing quartzite. This length was covered in five stages of consolidation.

In April 1951, the section from G12 to C1 being completed, grouting operations could begin from upstream to downstream. After one cycle of operations, the headings were sufficiently close together for one heading to be shut down, the work being completed from the downstream heading.

The different stages were as shown in Table 1.

Acknowledgements

The consulting Engineer for the dam was Mr André Coyne, President of the International Commission on Large Dams, head of the Consulting Engineering Office André Coyne and A. Belier, and the Designer for the plant of the dam was Mr Henri Guerrier, head of the design office of L'Entreprise Industrielle.

The Contractor was L'Entreprise Industrielle. The Author was one of the Managers of the Contractor's head office in Paris.

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The Paper is accompanied by ten photographs and ten sheets of drawings, from some of which the half-tone page plates, the Figures in the text, and folding Plates 1 and 2 have been prepared, and by the following Appendices.

APPENDIX 1

TABLE 2.—CONCRETE OUTPUT AT TIGNES DAM DURING 1948 TO 1952

Note.—Figures in columns (2), (3), and (4) are correct only to nearest hundred.

Year	Month	(1) Output of mobile mixers : cubic yards	(2) Output of concrete plant : cubic yards	(3) Monthly output : cubic yards	(4) Yearly output : cubic yards
1948	—	5,400	—	—	5,400
1949	—	—	—	—	—
1950	May–June	846	9,300	10,100	—
	July	—	9,700	9,700	—
	August	75	17,300	17,400	—
	September	26	29,700	29,700	—
	October	153	32,300	32,500	—
	November	—	2,300	2,300	—
	December	286	4,900	5,200	—
	Total :	1,389	105,500		106,900
1951	January	795	—	800	—
	Feb.–March	123	—	100	—
	April	—	4,200	4,200	—
	May	432	24,000	24,400	—
	June	320	57,100	57,400	—
	July	1,104	78,100	79,200	—
	August	1,061	81,900	83,000	—
	September	1,429	74,300	75,700	—
	October	722	75,100	75,800	—
	Total :	5,986	394,700		400,700
1952	January	663	—	700	—
	February	299	—	300	—
	March	179	—	200	—
	April	—	11,400	11,400	—
	May	—	79,200	79,200	—
	June	—	61,100	61,100	—
	July	—	72,000	72,000	—
	August	—	49,900	49,900	—
	September	—	31,400	31,400	—
	October	—	7,400	7,400	—
	Total :	1,141	312,400		313,600

1948 :	5,400
1949 :	—
1950 :	106,900
1951 :	400,700
1952 :	313,600
	826,600 cubic yards

TABLE 3

Days of placing more than :	1951	1952
5,200 cubic yards (4,000 cubic metres) . .	0	2
4,600 „ (3,500 „) . .	1	4
3,900 „ (3,000 „) . .	8	11
3,300 „ (2,500 „) . .	34	27

APPENDIX 2

CIVIL ENGINEERING PLANT FOR THE DAM

Quarry

2 Bucyrus 29-ton well-drills
2 P. & H. 3-cubic-yard diesel shovels
2 Bucyrus 1·5-cubic-yard diesel shovels
2 Caterpillar angledozers D.8
10 Euclid 12-cubic-yard rear dumpers
1 Allis-Chalmers 42-inch gyratory crusher
6 Ingersoll-Rand wagon-drills

Aggregate Transport

2 200-ton-per-hour ropeways

Crushing Plant

2 parallel plants each comprising :—
1 Dragon gyratory crusher G.P. 32
2 „ „ „ G.S. 5
4 „ crushing rollers G.L. 100
1 Allis-Chalmers vibrating screen for 10-inch aggregate
1 Dragon „ „ V.N.R. 120
2 „ „ „ V.N. 120

Stockpile

5 silos, each 5,000 tons capacity

Batching Plant

Johnson batching and mixing plant with four Koehring-Winget 3-cubic-yard mixers

Concrete Transport

4 60-ton standard-gauge bogie-wagons hauled by four diesel-electric locomotives of 80 horse-power
16 Garbo 6-cubic-yard buckets
2 Lidgerwood 20-ton Blondin cableways
1 „ 10-ton „ cableway (used as a general-purpose cableway)

Concrete Placing

Blaw-Knox steel forms (French manufacture)
30 Chicago-Pneumatic electric vibrators

Compressed-Air Supply

4 Canadian Ingersoll-Rand compressors, 126 horse-power
4 „ „ „ 203 „
1 Spiros compressor, 180 horse-power

Miscellaneous

1 Bucyrus 1·5-cubic-yard diesel shovel
1 P. & H. 1·5- „ „ „
1 Lima 2- „ „ „
1 Marion 1- „ „ „
2 Nord-Est 300-litre „ shovels
1 Pinguelly 600- „ „ shovel
3 Caterpillar angledozers D.8
2 „ motorgraders D.12
4 Muir Hill 3-cubic-yard dumpers
25 general-purpose lorries
Pumps, cranes, mobile mixers, etc.

Cement Transport

From the cement plant at Le Teil, on the south bank of the Rhône, to the nearest railway station at Bourg-St-Maurice, by 91 private bulk-cement wagons, hauled by special trains by the French National Railways.

From Bourg-St-Maurice to the dam site, a 35-ton-per-hour single cement ropeway

APPENDIX 3

Statistics of Tignes Dam Construction

Man-hours worked	5,000,000
Constructional steel used	6,000 tons
Reinforcing „ „	500 „
Cement	162,500 „
Electric power consumed (in 4 years)	50,000,000 kilowatt-hours
Total area of workshops	2,200 square feet
„ „ stores buildings	5,000 „
Temporary roads laid	5 miles
Re-located roads	8 „
Accommodation and facilities for workmen	250 houses
	2,500 beds
	8 canteens
	4 first-aid stations
	1 recreation centre

APPENDIX 4

Installation of Adits Nos 12, 13, 14, and 16

Adit No. 12.—This adit was an extension of No. 13, through which compressed air and supplies were brought.

Adit No. 13.—The installation included :—

Access road
Camp with accommodation for 400 workers
Cottages for the staff
1½-ton reversible ropeway for supplies and canteen catering
50-ton-per-hour double ropeway on steel towers, for aggregate transport
20- „ „ „ „ cement transport
500-horse-power compressor plant
3 100- „ ventilating plants
2 battery rectifier houses
Machine shops
Sheds for locomotives and electric shovels, storage silos for aggregates with steam heating system for winter time

Adit No. 14.—This adit was practically only accessible by a 3-ton reversible ropeway, hauling personnel and supplies. The installation included :—

250-horse-power compressor
2 100- „ fans
Battery rectifier house
Various workshops
Field office

Adit No. 16.—A military road was used as access, and had to be rebuilt twice because of heavy traffic. The installation for this adit was similar to that for adit No. 13, but also included :—

1½-ton reversible ropeway for supplies and canteen catering
50-ton double ropeway for aggregate with two unloading stations, one for the adit, the other for the surge tank.

Equipment used for Driving the Malgovert Tunnel

60-centimetre-(2-foot-) gauge rolling stock	
Card Iron 2½-cubic-yard wagons	45
4/6-cubic-yard tilting wagons	40
1-cubic-yard tilting wagons	300
1½- „ „ „	110
3 „ hopper „	15
Battery locomotives, 16 horse-power	6
„ „ 22 „	4
Diesel „ 50 „	16
Prefabricated tracks with 19-lb. per-yard rails	4,900 yards
Eimco shovels No. 21	10
Metre-gauge rolling stock	
Austin Western 5-cubic-yard wagons	75
Flat 4-wheel wagons carrying 2 containers	20
„ „ „ 3 „	50
Flat bogie wagons carrying a concrete plant or a drilling plant	10
Battery locomotives, 30 horse-power	16
Diesel „ 70 „	8
Steam „	2
Rail, 18 kg. per metre (36 lb. per yard)	74,000 yards
Eimco shovel No. 40 *	6
Various plant and equipment	
Drifter DA. 35	50
Hand jack-hammers	160
Drilling jumbos for 8 drifters	2
„ „ 4 „	2
Jack legs	30
Chipping hammers and pneumatic diggers	150
Paving breakers	80
2-inch and 3-inch pipes for water supply	13,200 yards
4- „ pipes for water and air supplies	30,000 „
6- „ „ „ air supply	22,000 „
300-millimetre air ventilation pipes	5,500 „
500- or 600-millimetre air ventilation pipes	10,000 „
Electric fans, 100 horse-power, 3 cubic metres per sec. under 1,000 millimetres water	10
Root pressure fans, 140 horse-power, 4 cubic metres per sec. under 1,800 millimetres water	2
Piston pumps, air driven, 100 cubic metres per hour	23

* American Conway 75 shovels were first used, but in compliance with a request from Electricité de France, they were re-sold to " L'Entreprise Borie " for the Isère-Arc tunnel, and these 6 Eimco shovels were ordered. Before they were available, 6 English Conway shovels were lent by Electricité de France.

Lump pumps I.R. No. 25, 30 cubic metres per hour	30
Centrifugal pumps, diesel-driven, 200 cubic metres per hour	9
Centrifugal pumps, low-pressure, from 10 to 40 cubic metres per hour	34
Centrifugal pumps, low-pressure, from 80 to 300 cubic metres per hour	19
Centrifugal pumps, low-pressure, 850 cubic metres per hour, electrically driven	6
High-pressure centrifugal pumps, 20 to 50 cubic metres per hour	11
„ „ „ Cameron class, 3-stage, 150 cubic metres per hour	6
Transformers, 15,000/220 volts, 10 to 30 kilovolt-amperes	11
„ „ „ 50 to 75 „	13
„ „ „ 100 „	26
„ „ „ 150 to 400 „	41
„ 3,000/220 „ 125 „	6
„ „ „ „ „ ironclad for use in galleries	6
Vertical compressors, single-stage, 100 horse-power, 360 cubic feet per min.	13
Vertical compressors, two-stage, 80 horse-power, 350 cubic feet per min.	4
Vertical compressors, two-stage, I.R. XV, 127 horse-power, 814 cubic feet per min.	15
Vertical compressors, two-stage, I.R. XV, 203 horse-power, 1,291 cubic feet per min.	8
Portable compressors, diesel driven, 315 cubic feet per min.	10
Lining plants with mixer and C.P.O.A.C. air placer	6
Vibrating forms for invert-pouring operations	2
Shutters for lining galleries	250 yards
Dollies for moving a 10-metre length of the above	5
Special hoist plants for reversible ropeway, 1-ton load with cable and cars	6
Diesel half-track trucks	8
Jeeps with trailers	40

Since they were owned partly by Electricité de France, the batching, screening, and crushing plants, the aggregate and cement ropeways, and the equipment used in the quarries and borrow-pits are not recorded in the above list.

APPENDIX 5

General Description of Method of Ground Consolidation Used for Driving Tunnel

Theory

Test boring and grouting.—The work is best pictured as a cylinder of ground concentric to the proposed boring. This cylinder is composed of fine crushed quartzites, the particles of which are perfectly uniformly graded.

During previous stages, the tunnel has been strengthened up to point c (see *Figs 16*) but has only been excavated up to point b, in order to have a protection of consolidated ground. A reinforced-concrete bulkhead wall ab provides protection against fissures in the ground, which might occur owing to the grouting pressure.

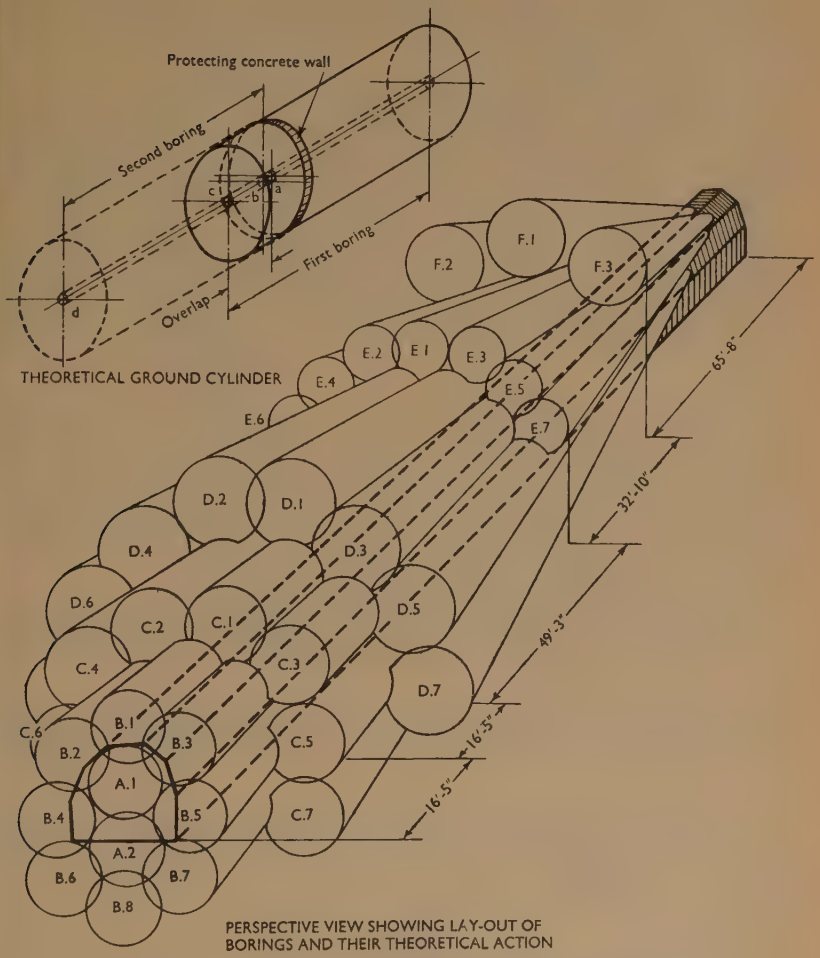
A 3-inch-diameter hole is first drilled through the virgin ground up to point c, and is lined with a pipe. This operation avoids interference with the ground already treated.

Boring (at 2·8 inches diameter) is then resumed, running the drill through the pipe, as far as point d. The length cd of virgin ground is then tested and treated as follows :—

(1) Particle-size is determined, and the permeability of the rock and the water flow are measured.
(2) Strengthening by grouting is carried out as necessary.
(3) The tunnel is driven as far as point d.

This cycle of operations is then repeated.

Figs 16



Method of Grouting for Tunnelling in Quartzite

Conic-frustum treatment.—The treatment of the ground is effective only up to a certain radius from the bore-hole. This radius depends on the pressure of the ground-water and other factors that cannot be measured. It is therefore determined empirically in order to ascertain the right grouting pressure.

Equipment Lay-out of a Grouting Job

The borings are made with four standard continuous-core drills working together. They are mounted on a vertically sliding platform and are able to bore from any angle. Behind these machines, the raw materials, mixers, and grouting machines are stored.

Quartzite Treatment

The most difficult stratum for grouting was theoretically the powdery sand. Water circulates in the sand under a pressure of 56½ lb. per square inch. The first operation was therefore to cut off this water circulation. After many tests had been carried out, four stages of treatment were found to be necessary in order to excavate with safety in these crushed sands.

(1) *Sodium silicate solution grouting.*—This initial process prepares the ground and makes the penetration of the second grouting easier.

(2) *Cement Grouting.*—Under the action of the grouting pressure, circulation passages are produced in the sand, and the cement acts like a wedge, compressing the quartzites. This grouting is continued until the pumps are producing a pressure of 1,432 lb. per square inch, and cannot force through any more cement. The ground is now divided by cement seams and has a low and fairly constant permeability.

(3) *Sodium silicate and phosphoric acid solution grouting.*—This low-viscosity solution can set after a given time, which can be regulated by adjusting the ratio of the constituent parts. (On this job, the setting time was 15 minutes.) A grouting pressure of up to 142·23 lb. per square inch was used. When this grout sets, it forms a consistent gel which coats the sand grains, dries the ground, and produces cohesion.

(4) *Cement grouting.*—This grouting gives perfect keying to the ground by further compression.

Quantities Used—Control of Operations

The tunnel to be excavated had a diameter of 19·6 feet and the consolidated ground was theoretically a cylinder with a diameter of 81 feet. This represents a cylinder of 5,160 square feet cross-section to be consolidated.

The purpose of the first cement grouting is to compress the ground, and the quantities to be used are limited by the maximum pressure determined. But the sodium silicate and phosphoric acid solution grouting has to fill the theoretical cylinder of the boring, and so the quantities to be injected must equal the cylinder volume multiplied by the voids ratio, which was 30 per cent.

The final keying is consequently limited by the grouting pressure.

As each length of treatment, in the form of a conical frustum, overlaps the last, the bore-holes have to penetrate the subsequent length and thus afford an excellent method of examining the treated ground before it is excavated.

Treatment of the Contact Areas

The treatment of the areas of ground in contact with the roof and walls was performed in the same manner, but consolidation was almost exclusively achieved by cement grouting.

Excavation through the Grouted Strata (*Fig. 17*, facing p. 505)

Normal methods were used successfully. The excavation was begun with pavement breakers, but the consolidated ground was so hard that it was found necessary to use jack-hammers and explosives. No excavation difficulties were encountered, nor did any fall of ground occur.

Statistics of Boring

Each stage, involving a 115-foot length of tunnel, required forty borings. The total figures for the job were :—

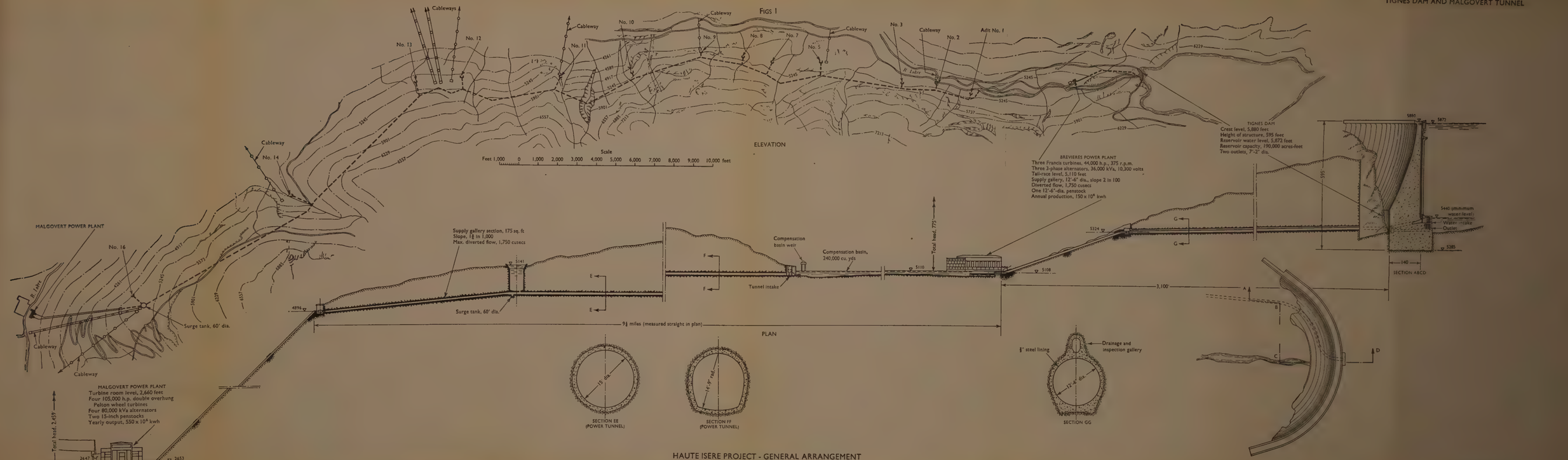
16,085 feet of boring and 3½ miles of re-boring ;
3,800 tons of cement and 5,240 cubic yards of gel used for grouting.

These figures demonstrate the large extent of the work carried out. The success of the job was largely attributable to the great care that was exercised throughout. The possibilities of the method are wide, and it is considered that it could often be applied where ordinary tunnelling methods failed or became unsafe.

THE CONSTRUCTION OF TIGNES DAM AND MALGOVERT TUNNEL

PLATE I
TIGNES DAM AND MALGOVERT TUNNEL

FIGS 1



HAUTE ISERE PROJECT - GENERAL ARRANGEMENT

JEAN PELLETIER

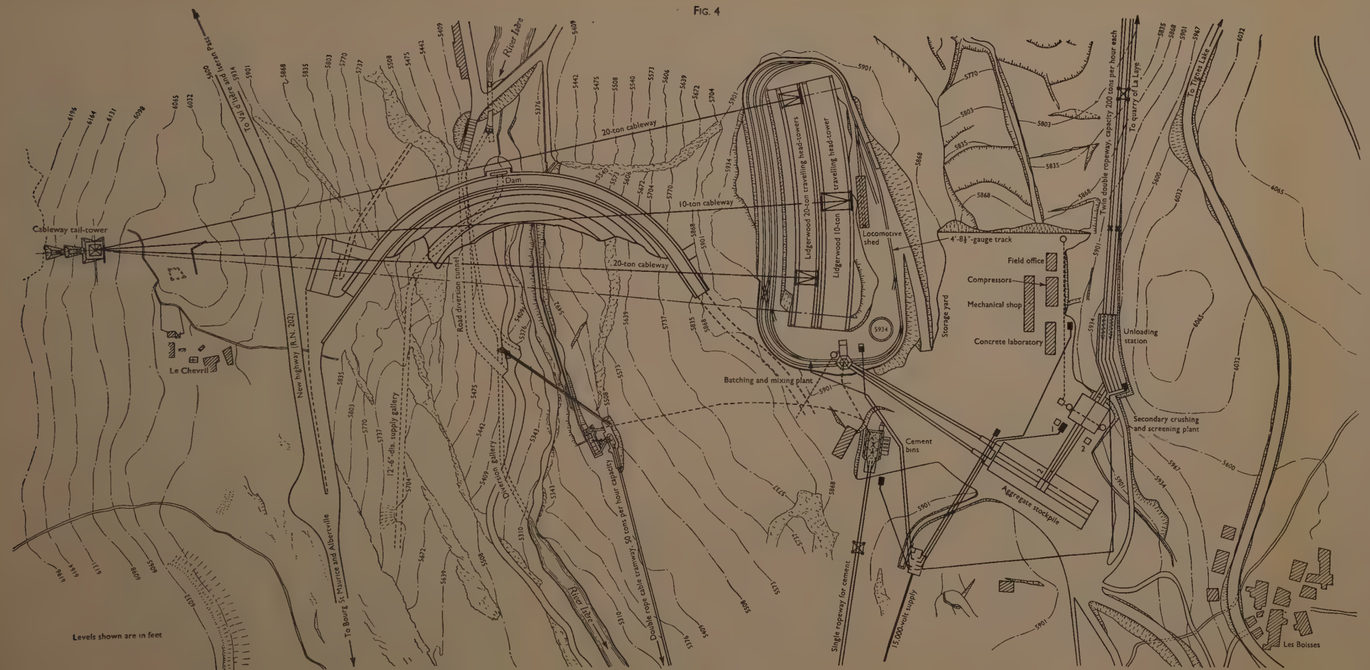
The Institution of Civil Engineers Proceedings, Part III, December 1953

WILLIAM CLOWES & SONS, LIMITED : LONDON

PLATE 2
TIGNES DAM AND MALGOVERT TUNNEL

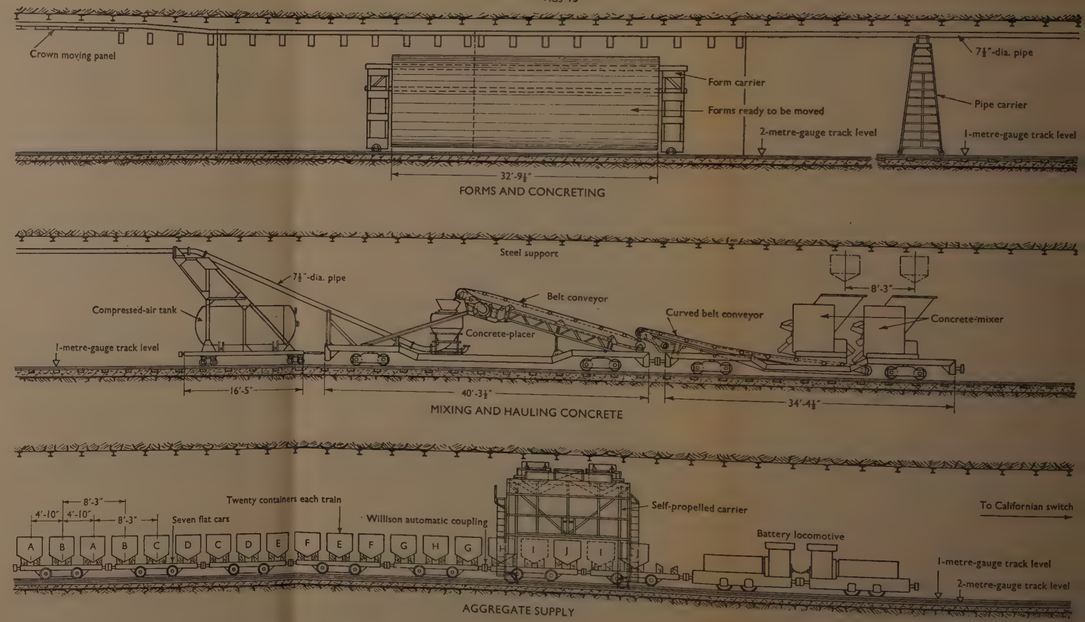
THE CONSTRUCTION OF TIGNES DAM AND MALGOVERT TUNNEL

FIG. 4



PLAN OF TIGNES DAM AND PLANT LAY-OUT

FIGS 13



Discussion

The Author introduced the Paper with the aid of two films.

Mr H. D. Morgan said that the dam was a typical one by Mr Coyne, but he was a little surprised to see that it was designed with concentric arches. That was not Mr Coyne's usual practice, and it would be interesting to know why he had done it. There might be some good reason for it of which the Author was aware.

The rate of placing the concrete was remarkable. The average daily output of 2,550 cubic yards with a maximum of 6,000 cubic yards was an excellent performance. It was notable to see from the film that vibrators had been used for moving the concrete during placing. That was a technique which was not always appreciated. Mr Morgan had noticed, as a member of the Committee of the Institution which dealt with vibration, that whilst the principal questions dealt with were quite properly those of obtaining maximum density and strength, the value of the immersion vibrator in assisting to place harsh mixes was sometimes overlooked. Dams such as the one under discussion were built in remote places, and it was necessary to take the aggregate as it was found. He recalled seeing concrete placed at the Castelo-do-Bode dam in Portugal, in which the aggregate was not by any means perfect, and without the use of vibrators it could not have been handled at all effectively.

The strength was stated to be about 2,400 lb. per square inch at 7 days, and that was fairly good for concrete with an aggregate which was difficult to handle, and therefore not favouring a low water/cement ratio.

With regard to making the test cylinders, which were 12 inches by 24 inches, he presumed that the 10-inch aggregate had been taken out first. That was a common practice, and he had seen at the Castelo-do-Bode dam a workman operating a very small hand mixer, and stopping to remove boulders, afterwards tipping out the remainder of the mix which was then put into the mould. The crushing stress was then multiplied by a reduction factor in order to arrive at the strength which it was thought would be obtained in the field when the boulders were kept in. Mr Morgan presumed that the cylinders at Tignes had been made in moulds and that the 10-inch and possibly even the 5-inch aggregate had been removed after mixing. He hardly imagined that a diamond coring machine was used, and even if it were he doubted whether the results would be of much value if the large aggregate were cut into. Cores 12 inches in diameter had in fact been taken recently on works for which his firm was responsible, but those were in smaller aggregate in a matrix of " Colgrout."

There did not appear to be any reference in the Paper to temperatures. With the concrete going in at such a rate one would imagine that something was done to measure the temperature internally, but nothing was said about it. It would be interesting to know something about that, and also

if any strain gauges were used to measure deformation after filling for comparison with calculated stresses.

Another point of great interest was the pipeline which had been wrapped with high-tensile wire and then expanded. The Malgovert pipelines were stated to have required 4,500 tons of steel treated by that process as compared with the 35,000 tons of ordinary steel that would otherwise have been necessary. That was a striking ratio and he was very interested in that technique.

It was obvious that the tunnel had been a work of very great difficulty. Judging from the film, he could only recall one tunnel which had been wetter than the Malgovert tunnel, and in that case it had been almost like being under a waterfall. The great amount of steel centering which had been put in, together with the information shown on the longitudinal sections, where methods of driving had been changing all the time, gave a clear indication of the difficulty of the work.

Of course, the engineer often did not know what he was facing in a tunnel until he got underground. However much he might study the geology and boring logs he did not know what he was up against until then, and it was difficult to write a specification and bill of quantities to try and eliminate as much as possible the so-called contractor's risk. It was sometimes impossible to make a tender which covered every possible eventuality without greatly inflating the amount. Reasonably competitive tenders did not always result in such circumstances. It would be interesting to learn how that problem had been dealt with.

Mr Oliver Dawson said that one of the points which stood out very well in the Paper was that concerning the difficulties caused by the location of the job. For example, there was the high altitude of 6,000 feet, and the site being situated in a narrow steep valley which had been very ingeniously crossed with frequent cableways in quite an alarming and remarkable manner. The problem of handling 160,000 tons of cement up to the site from the railway station itself had surely been a great one, but it had again been solved by long cableways spanning the valley.

The scarcity of plant at the beginning of the job was shown by the use of donkeys for hauling the trains. It seemed a long way from donkeys to the list of plant which appeared at the end of the Paper, but that was a credit to the French people for the way in which they had recovered after the 1939-45 war.

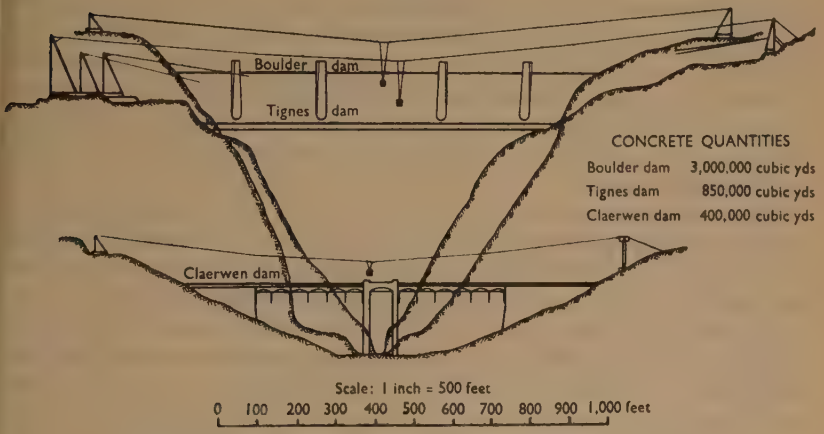
The ground through which the tunnel was driven had surely given everyone a considerable headache, although possibly they had had some knowledge of what to expect. The mountain was called the "Rotten Mountain," and it would certainly appear to have lived up to its name! It was noticed that pneumatic placers had been used and they had presumably been justified. It would be interesting to know why bulk batching was used for the materials in the tunnel as compared with weigh-batching for materials on the dam plant.

It was noticed from the Paper that the aggregates were up to 10 inches in size and that difficulty had been experienced in handling and placing the concrete although the vibrators were of the largest size made. In his view, many of the vibrators available were little more than toys, and there was scope for the production of proper vibrators.

There appeared to be little reference in the Paper to the excavation of the main dam foundations and the diversion of the river from the road, and it would be interesting to know if any difficulty had been encountered in that connexion and if the dam had been grouted in any way.

In conclusion, he showed a slide (reproduced in *Fig. 18*) depicting the Claerwen, Tignes, and Boulder dams to the same scale. The Claerwen

Fig. 18


Boulder dam
Tignes dam
CONCRETE QUANTITIES
Boulder dam 3,000,000 cubic yds
Tignes dam 850,000 cubic yds
Claerwen dam 400,000 cubic yds
Claerwen dam
Scale: 1 inch = 500 feet
0 100 200 300 400 500 600 700 800 900 1,000 feet

COMPARISON OF TIGNES, BOULDER, AND CLAERWEN DAMS

Dam was the largest in the British Isles, and the Figure illustrated very clearly the relative efficiencies of the valleys in the three countries for the construction of dams. The valleys in Great Britain were flat and shallow whereas the Tignes valley, which was a very narrow gorge, was almost the perfect shape for the dam. The Boulder canyon was wide and large at the base, and that to some extent accounted for the difference in the quantity of concrete used—3,000,000 cubic yards for the Boulder dam and 850,000 cubic yards for Tignes.

The three dams were built using cableways of roughly similar spans. Of course, the cableway in the case of the Claerwen Dam was of 10 tons capacity, whilst the cableways of the other two dams were of 20 tons capacity. The relative use of that particular piece of plant in the construction of dams could be observed.

Dr Charles Jaeger observed that, on p. 485, it was stated that the crushing plant had produced five grades of aggregate from 0 to 10 inches,

which had been graded continuously. In the paragraph headed " Concrete for the Dam," it was stated that the concrete had been gap-graded and that the 2- to 5-inch sizes had been suppressed. Had that material had to pass through the crusher a second time ?

He also asked if the decision to use a gap-graded concrete had been taken on the basis of theoretical considerations by the designer of the dam, or whether it had been taken by the contractor because it was more convenient for the type of aggregate produced by the crushing plant. Was such an aggregate not dearer than an ordinary continuous aggregate ? Were not 10-inch boulders very heavy for the existing concreting plant available on the market ? He would also be grateful for additional information on the dust cleaners used for the removal of the excess dust.

On p. 481, a short description was given of the two pipelines designed and manufactured by Bouchayer and Viallet. It appeared that the pipeline was divided into three sections—a plain steel pipe, a prestressed pipe with steel bands, and a prestressed pipe with cable reinforcements. It would be interesting to know the heads and the pressure on those three sections. The steel used at Malgovert was said to have an ultimate strength of 79,600 lb. per square inch ; did that apply to all three sections or only to the prestressed pipes ?

Mr P. B. Nissen said that the success with which three radial cableways had been operated was interesting, especially in that the smaller, general-purpose cableway could cross over the others to cover the same ground at the same time. That was a vital factor in the speedy and efficient construction of a high dam, since poor disposition and coverage by cableways could easily result in a bottleneck in the supply of materials to the dam site. He had experienced that sort of thing in the case of twin cableways, each with its own synchronized head- and tail-tower working on lines parallel to each other and the gorge, on a smaller arch-dam project, and he had found that it often proved to be a considerable embarrassment to be obliged to request the interruption of the concrete placing programme in order to supply some small yet vital piece of equipment to another part of the site. He knew that the operation of only two cableways required careful day-to-day planning and close co-operation between remote sections of the works if costly and time-consuming delays were to be avoided. The provision and location of the three cableways at Tignes had surely given the contractor a great deal of satisfaction and confidence.

The temperature control of setting concrete within the mass of a large dam was always a major construction problem, and a very costly one—especially when a forced water-cooling system was used to reduce the temperature of the mass and ensure adequate shrinkage before grouting the contraction joints. It was interesting to note that no such system was in evidence at Tignes, whereas at the dam to which he had referred earlier, there had been a very elaborate system of cooling pipes and refrigeration plant for the comparatively small volume of 180,000 cubic yards. Perhaps

the Author could give an indication of the temperatures of the concrete mix at placing, and also say something about the method and magnitude of the temperature and shrinkage observations on the dam before grouting. Could the Author also give some details of the system and an indication of the success of the contraction-joint grouting itself ?

Mr J. D. Gwynn said that one of the points which he had particularly noted in the Paper was the high speed of the cableways which had been selected for the construction of the dam. The speed was considerably higher, particularly in travelling, than was the normal British practice. In analysing the performance of the cableway in detail, travelling time was not by any means the only factor. It did not form a very large percentage of the whole, and after giving details of the cableway specification, the Author had made a brief statement in which he had shown the high penalty which was paid in the necessary changing of button ropes and travelling ropes owing to the high-speed operation. It was a very much higher rate of change than was found necessary with British practice. It would be interesting to know whether that high speed had been specially selected after consideration, or whether it was that which Lidgerwood's, who specialized in cableways, had to offer as their normal job.

With regard to tunnel supports, the Author had shown in the film a crippled piece of tunnel rib—crippled at the beam and not at the joint. In the collapse which had occurred it would be interesting to know whether failure had ever resulted at the joints or whether it had been associated usually with the beam in between. That was of great interest, because the tunnelling conditions experienced by the Author had evidently been among the most extreme it was possible to have, and calculation of tunnel-rib stresses was so full of supposition that any actual results of failure were very useful.

Mr J. K. Hunter after referring to the formidable difficulties encountered in driving the Malgovert tunnel, described another case in which a tunnel had run into serious trouble, from which the contractor had eventually extricated himself with unexpected ease.

About 2 years previously, a tunnel had been under construction through the Kharkunan mountains, in Persia, which separated the hinterland from the Persian Gulf. That tunnel was about 2·8 kilometres long and 10 feet in diameter. After a slow start, the contractor had got going fairly well when one day, whilst drilling the face, one of the drill rods had suddenly shot out with considerable violence, to be followed by a high-pressure jet. A cavity containing water under considerable pressure had been struck.

It had been hoped that the cavity would very soon drain itself and that the water pressure would die away. However, after about 4 or 5 months the head of water behind the face showed no sign of diminishing. The contractor had eventually been persuaded to build a substantial concrete bulkhead about 60 feet back from the face, and he had then made preparations for drilling the face under extremely bad working conditions. Having

loaded the face, everybody had withdrawn and waited with some trepidation for the expected onrush of water. The whole thing had ended in rather an anticlimax, for after the face had been blown, the amount of water which escaped had not increased at all—in fact, after an hour or two, it had diminished considerably—and the contractor had been able to continue the driving operations with no greater difficulty than he had experienced previously. After 3 or 4 months the two headings had met and his difficulties were over.

That tunnel, after 4 years of rather hazardous and often frustrating work, was on the point of being completed, and in 2 or 3 months' time the waters of the Karun river which now ran into the Persian Gulf would be diverted into the interior of the country for irrigation purposes.

The Author, in reply, observed that Mr Coyne had probably decided on a design with concentric arches, first, because of the dam site, and secondly, to simplify the shuttering, especially on the upstream face.

Mr Morgan was correct in assuming that the 10-inch aggregate had been removed from the concrete for the test cylinders. Cores 12 inches in diameter had been taken out of the dam itself, and tests on them had confirmed the results of the preliminary concrete tests.

Temperatures had been recorded in many parts of the dam, and the readings had been normal. An acoustic strain-gauge system had also been used. Since the dam had been filled only in 1953, Electricité de France could not yet publish a comparison between the deformation after filling and the calculated stresses.

The Author could not give any further information on the pipe-lines because they had not been built by his firm, and he did not know how they had been designed. He could state, however, that if they had been constructed in 1930 they would have required 24,000 tons of steel, as compared with the 4,500 tons actually used.

Mr Dawson had asked why bulk-batching had been used for the materials in the tunnel lining, as compared with weigh-batching for materials on the dam plant. The Author replied that weigh-batching plant had not been available in time to incorporate it in the tunnel-concreting plant. Most of the equipment had, in fact, been selected just after the war, when there was a shortage of all kinds of plant.

The main feature of the plant used was the tilting volumetric dosing apparatus, illustrated in *Fig. 19.* The hole in the bottom of the aggregate hopper was opened and closed by movement of the discharge hopper.

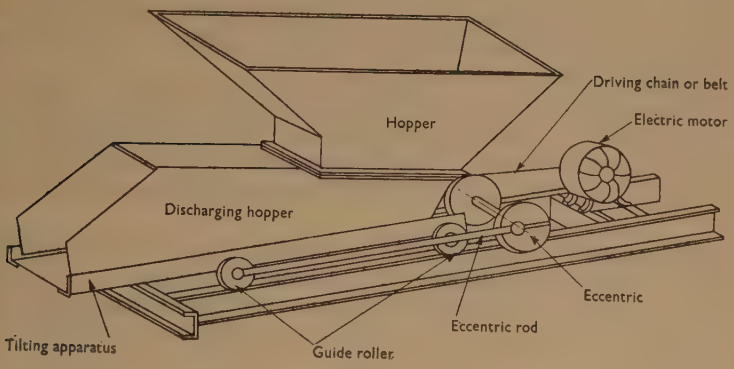
The box was moved by means of an eccentric, which controlled the number of cycles per minute. The eccentric was driven at a uniform speed by an electric motor.

Mr Dawson had also asked if any difficulties had been encountered in driving the two diversion tunnels. As mentioned in the Paper, those tunnels had been driven through very hard quartzite. Carbide-tipped drill rods had to be scrapped after driving 80 feet, and being sharpened

five times. That rock, however, had given good watertightness for the abutments.

With regard to the difficulties of excavation for the dam, the Author stated that excavation for the abutments had been carried out in the winter-time. It had been necessary to excavate a total of 100,000 cubic metres for the base and abutments of the dam. The work had been hampered by the lack of access roads.

Fig. 19



Grouting of the abutments was carried out as follows :—

(*a*) A deep cut-off was provided by 100-foot grout-holes.

(*b*) A complete grouting zone was established along the downstream side of the left abutment, where there were some slightly crushed quartzites.

(*c*) A similar zone was formed at the right abutment around the two diversion tunnels.

(*d*) Two lines of 16-foot grout-holes were drilled for cut-offs along the upstream abutment and inside the dam.

(*e*) The grouting of the joint between the concrete of the dam and the rock was done by means of pebble pockets located at the surface of the rock and connected to the upstream and downstream faces by a grout-pipe.

In reply to Dr Jaeger's question concerning the suppressed aggregate sizes, the Author stated that part of the 2- to 5-inch aggregate had been recrushed in a secondary crushing plant to be used as sand, fine aggregate, or coarse aggregate for the face concrete—the latter being gap-graded between $\frac{3}{8}$ and 2 inches. The decision to use gap-graded aggregate had been based on theoretical considerations, but the Author agreed that it was more expensive than ordinary continuous aggregate.

Sturtevant air-cleaners had been installed inside the crushing plant, but it had later been found necessary to supplement them with some Italian air-cleaning equipment.

Referring to the 10-inch aggregate, the Author said that it had been too heavy for the plant, and it had been necessary to modify the mixers and the screen, and many other parts of the plant, particularly the batching equipment. Although the 10-inch aggregate had caused many difficulties, the contractor had been obliged to use it because the French authorities had wanted to build the first dam with the concrete made entirely in the mixers, including the 10-inch aggregate. It had been thought that £100,000 might be saved in that way, but that had not been the case, for nearly the same amount had been spent in modifications to the plant and on its maintenance.

The cableways, which were of a standard type, had been selected for their high speed of travel. A new type had since been developed by the cableway constructors which could be used for travelling and hoisting simultaneously, by means of two separate motors, and the speeds were even faster.

As Mr Nossen had pointed out, the third (10-ton) cableway had been extremely useful, especially since it had been able to cross over the two 20-ton cableways. That difficult manœuvre had been carried out several times a day, but only one accident had occurred through failure of the cableways.

The average temperature of the concrete when placed was 59° F.

With regard to grouting systems, Mr André Coyne had specified a primary and secondary system similar to those used for the Boulder Dam. The contraction joint grouting was very successful.

Mr Gwynn had raised a question concerning the steel supports in the tunnel. Polygonal frames had been used because, in France at that time, it had been impossible to obtain a steel section that could be bent. Towards the end of the job, curved steel supports had been used for the crown, and had been found more satisfactory than polygonal supports. The failures in those polygonal frames had generally occurred in the beams and not at the joints, which had been designed to be as strong as the beam itself in bending. The joints had embodied butt-plates welded to the beam ends and reinforced against bending by welded gussets, and were assembled with high-tensile-steel bolts.

The Author observed that there were many similarities between the case Mr Hunter had described and the difficulties encountered when driving the upward heading of adit No. 16 for the Malgovert tunnel.

In November 1950, at about 1,000 yards from the entrance, while drilling some trial holes a few yards from the face, a big squeeze had appeared in the crown near the face. That had caused failure of some of the steel supports, and the crown had suddenly collapsed, nearly blocking the gallery. Water flooded the gallery, and rose in a few minutes to a level

of more than 3 feet. After a few days, the water level stabilized itself at a little less than 2 feet.

As in Mr Hunter's experience, the tunnel had run into a cavity containing water under pressure. After a few weeks, the flood not having subsided, it had been decided to continue through the fallen rock by forepoling methods, using old rails as forepoles. The cavity was successfully passed, but 5 months had elapsed before it had been possible to resume driving the tunnel. The heading was so wet that the crew had had to work under very uncomfortable conditions, which had continued until the downstream heading was encountered.

The inflow had not actually increased, but had been distributed all along the new tunnel. The worst flood had been about 4,000 gallons per minute. Electrical pumps had been used to dispose of the water.

In reply to Dr Jaeger's question regarding the Malgovert pipelines, the Author stated that the head pipe, 960 feet long, was of welded steel and was subjected to a maximum head of 308 feet. On the end of that section, the pipeline bifurcated into two sections, each of which consisted of:

(*a*) a length of 5,100 feet with high-tensile steel circular hoops, under a maximum head of 2,630 feet, followed by

(*b*) a length of 1,000 feet in rigid steel hoops, under a maximum head of 2,866 feet.

HYDRAULICS ENGINEERING DIVISION MEETING

28 May, 1953

Sir Claude Inglis, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Hydraulics Paper No. 1

"The Lynmouth Flood of August 1952"

by

Charles Herbert Dobbie, B.Sc.(Eng.), M.I.C.E., and Peter Otto Wolf, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

Part 1

The Lynmouth floods of August 1952 were investigated before the evidence was destroyed, and assessments of the flow in different parts of the Lyn River system are put forward based on this evidence. The selection of values for the roughness coefficient in Manning's formula is a controversial matter, but an overall check was provided by the figures fitting together into a reasonable pattern.

Since it is unusual to find a torrential river in England, some special topographical features are obviously involved. The explanation of geomorphologists is that on a coast showing the highest cliffs of England, the sea eroded away the valley side and captured the river. The main course, the East Lyn River, has been graded to a steep profile but the tributaries have not been equally graded and have been left hanging. For the West Lyn River, joining the East Lyn nearest the sea at Lynmouth, the effect is greatest and the profile is precipitous.

In flood, the rivers attain very high velocities and carry heavy bed loads. Remedial works recommended follow European practice for torrential rivers.

Part 2

The meteorological conditions leading up to the catastrophic rainfall of the 15–16th August, 1952, are briefly discussed, against a background of the average long-term conditions in the area. The rainfall on that day, and the resulting run-off, are described in relation to the geology, topography, and vegetative cover of the catchment.

The run-off, as computed from site surveys, is compared with values calculated in accordance with the Interim Report of 1933 of the Institution Flood Committee.

Mention is made of the obstruction of the natural channels of the Lyn rivers and their tributaries by dead vegetation and boulders and by recent building work.

The scale model at the Imperial College, of a short section of the West Lyn suitable as a gauging site, is described. With its aid, a stage-discharge curve has been determined which will enable the site to be used as a permanent gauging station. This curve, together with a record of high-water marks, has yielded a value for the peak flood intensity during the night of the 15–16th August.

Part 1.—Measurements and Geomorphology
by Mr Dobbie

Introduction

The Lynmouth Floods of August 1952 were investigated before the evidence was destroyed. In the absence of gaugings, assessments of flow based on that evidence were developed for each section of the Lyn river system. The method used was to take longitudinal and transverse sections of selected reaches and to apply Manning's formula. The selection of coefficients was a critical operation but an overall check was obtained by the figures fitting together into a reasonable pattern. A further check, independent of such coefficients, was obtained by a model-study of one particular site.

Since it is unusual to find a torrential river in England, some special topographical features are obviously involved. A simplified explanation is that on a coast with the highest cliffs in England the sea eroded away the valley side to capture the river. The main course, the East Lyn river, has been graded back to a steep profile but the tributaries have not yet been equally graded and have been left hanging. For the West Lyn river, the tributary joining nearest the sea in Lynmouth, the effect is greatest and the bed is indeed precipitous. In flood, the rivers attain high velocities and carry heavy bed loads. Remedial works recommended follow European practice for torrential rivers.

Measurement

The assessment of peak flows in each part of the Lyn river system during the flood was the first hydraulic investigation, commenced a fortnight after the occurrence. There had been no gaugings. Steps were taken quickly to record measurements from the evidence left by the floods before it became obliterated. The value of the evidence had to be weighed. The wrack line giving the top level may have been washed down by side flow after the passing of the peak flow, or, on the other hand, might represent only a transient surge. The bed as left giving the bottom level may have been scoured or built up subsequent to the peak flow.

There was no novelty in the methods adopted, which would suggest themselves to any hydraulic engineer. Walking surveys of the river bed had already been made by engineers of the Ministry of Agriculture and of neighbouring River Boards. From this information, gauging sites were selected and surveyed. At each site four cross-sections were taken, the gradient of the wrack line was measured, and all were related to Ordnance Datum. This procedure was somewhat laborious at many of the sites.

Theoretically, further sites could be added but, in practice, the first selection had to suffice. The sites were selected for straightness in plan, uniformity in cross-section, and reasonableness of access. By inspection, lengths were eliminated where there were doubts about surges or about large bed changes.

While survey of the sites was in progress, each was visited by a Polish engineer with experience of torrential rivers and he selected appropriate values of the coefficient n to be used in Manning's formula. This formula was adopted as being easy for calculation and as being the standard for both the Ministry of Agriculture and the United States Department of Agriculture. The value of the coefficient adopted is of paramount importance, making a large difference to the result, as academic critics of so-called " Coefficient hydraulics " are quick to claim. No practical substitute, however, appears to be available. The American National Resources Committee, in their publication *Low Dams*,[1] set out by photograph and description a series of rivers having coefficients varying from 0·035 to 0·150. For the Lyn river system, values 0·045 to 0·08 were used. On some sites the centre channel was taken as 0·045 and the overspills on the sides as 0·06. The meadows were divided by frequent hedges and small woods, so, although some sections were clear, the higher values were taken because of the general level of obstruction.

The first check to be obtained was that the flow in confluence was approximately equal to the sum of the flows of the constituent streams. The check applied throughout the systems gave a good degree of consistency. There still remained the doubt that all the values might be consistently high or low, so a further check was desirable, independent of Manning's coefficient.

This was achieved by a model study of a peculiar rocky cleft in the West Lyn river which ended in a large waterfall, giving the advantage of a free discharge. A reasonable check was obtained and the values of flow were put forward without modification. (See *Fig. 7.*) Further sites are available for model studies and, over a period, stage-discharge diagrams could be built up, checked, and extrapolated to flood levels; but time and expense will not permit of such refinements. Flood flows are infrequent and special care would have to be taken, otherwise measurements could be extremely hazardous to instruments and observers.

Geomorphology

The study of land forms—a combination of geology and geography, using data from meteorology, hydrology, ecology, and even hydraulics and strength of materials—now widely known as geomorphology, adds yet another word to the esoteric vocabulary of the Civil Engineer, all of whose

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[1] The references are given on p. 546.

works have such a background. In dealing with part of a river system, some knowledge of the whole is desirable and understanding will be incomplete without some reconstruction of the past. To geomorphologists the longitudinal sections or profiles of a river system are the key to its development. A river tends to flow at a gentle slope getting flatter towards the sea and being steeper at the headwaters. In uniform stratum and under uniform conditions its profile would be a logarithmic curve and, in practice, if a river is left alone, it does grade itself reasonably to such a curve and the tributaries grade in similarly to the main course. The drainage pattern is, however, always complicated by other factors, since apart from earth movements there are changes in sea level affecting the outfall, there is piracy amongst river systems whereby when the watershed is eroded parts of one system are captured by the adjacent system, and there are the effects of glaciation. Despite these complications, remnants of past curves can be recognized and by extrapolation old courses can be plotted.

Most of the upper or middle reaches of English rivers grade to run out to a sea 600 feet above the present sea level. The lower reaches have thus been steepened or rejuvenated by this lowering of the outfall but since it has taken place over a long period the changes and gradients are eased. The change has not been a simple lowering of sea level relative to the land, but a series of oscillations with lowering predominant. The last sizeable movement was in reverse, a rise of sea level of 75 to 150 feet in post-glacial times, drowning many river mouths and leaving their buried outfall channels in the sea bed.

These processes can also be followed on cross-sections, for the lower reach of a rejuvenated river shows a valley within a valley and side terraces at the junctures. Frequently, a number of terraces can be found by shape and by content.

Fig. 1, facing p. 528, illustrates the bare plateau of Exmoor with its leached moorland soils, covered with heather, rushes, and grass. The immaturity of the upper reaches of the valley shows in the steep slopes, the interlocking spurs, and the incised watercourses.

Fig. 2 shows the confluence of the West Lyn river with its tributary at Barbrook. The western tributary, in the foreground, has a greater catchment area than the named river. A new combe, or washout, can be seen in the centre foreground. In the distance is shown the bleak upper plateau of Exmoor illustrated in *Fig. 1*. The hilltops are now cultivated land with shallow stony medium-to-light loam soils. The valley rides are afforested. The valleys are more open, with meandering streams in meadows showing some maturity.

When the profiles of the Lyn river system are examined (Figs. 5, Plate 1), it is at once apparent that rejuvenation of the lower reaches has been violent.

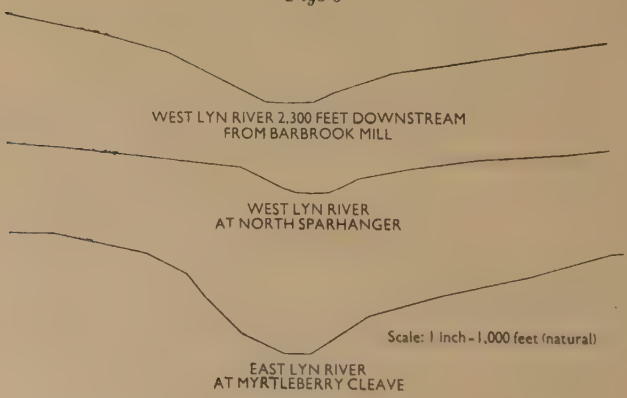
No appraisal of the whole development of the river system is to be attempted, for this would lead into controversial discussion well removed

from engineering ; instead, a simplified version is set out to show the main features and it is felt that without some understanding of these matters and of how Lynmouth came to an increment of its geomorphological fate, the study of the local hydraulics would be but a sterile exercise. Balchin[2] sets out the flattish surfaces of the land as seven platforms of marine erosion and it is over one of the cliffs so formed that the top waters of the river fall. A. Arber[3] has discussed the origins of the hog-backed cliffs peculiar to the north-Devon coast. It is, however, in the work of Scott Simpson[4] that is found the information relating to the violence of the Lyn river system.

He has investigated in detail the underlying cause mentioned by E. A. N. Arber[5] and others. It is that the sea has broken through the valley side of the East Lyn river and captured the river at Lynmouth.[4] There have obviously been previous captures and today erosion is in progress to the east, which will some day result in a further capture diverting the East Lyn river from Lynmouth. It will, of course, take a long time on a human life scale.

To the west, elevated remnants of the previous course remain in the famous Valley of Rocks, at Lee Bay, and at Woody Bay. The extrapolated profile of the West Lyn river lines up accurately with these remnants and the extrapolated profiles of the tributaries line up with such a main river profile. The last capture at Lynmouth, which is the dominant feature of the present drainage, took place when the sea was not more than 100 feet above its present level. After capture, river erosions would be very great and the regrading of the lower reaches would proceed vigorously. The East Lyn river, running along the strike in softish slates between thick layers of more resistant sandstone, has been graded to some extent although remaining youthful and turbulent. Its middle reaches have a gradient of 1 in 63 and that of the lower reach from Watersmeet to the sea is 1 in 27.

Figs 6


WEST LYN RIVER 2,300 FEET DOWNSTREAM
FROM BARBROOK MILL
WEST LYN RIVER
AT NORTH SPARHANGER
Scale: 1 inch - 1,000 feet (natural)
EAST LYN RIVER
AT MYRTLEBERRY CLEAVE

Cross-Sections

The tributaries with less flow and more resistant paths have failed to keep pace with this grading and are now discordant or hanging. The Hoaroak water ends in waterfalls in a precipitous course to Watersmeet. The West Lyn, the tributary nearest the sea, has a deeply incised valley, falling at 1 in 5·4 from the middle reach of 1 in 26. (See Figs 5, Plate 1.) The steepness of the valley sides is shown in the cross-sections plotted to a natural scale (see *Figs 6*).

Regrading now continues, the valley floors and sides being rapidly eroded and the products transported seawards by the rivers. The metamorphosed rocks with cleavage planes at a high angle to bedding planes break up into boulders of all sizes, and into smaller fragments and sand. Transport is small during ordinary flows and depends upon flood flows for real effect. In extreme flood, much material is moved and the valley sides are loosened up so that ordinary floods carry a good load in following years. The boulders in flood batter their way along and when joined by washed-out trees form temporary dams which burst with huge surges, moving further material.

Super-critical velocities of 20–30 feet per second are common and move great masses, for, according to Einstein,[6] a velocity of 15 feet per second will move a rock measuring 3 feet by 3 feet by 3 feet. After the August flood, a boulder weighing 7½ tons was found in the basement of an hotel.

The very violence of contemporary erosion gives the valleys great beauty. The walks along these torrential rivers have entranced Southey, Shelley, Coleridge, Wordsworth, and many others, and will undoubtedly continue to do so. To build in such a valley was natural enough, for few English beauty spots escape such attention. In the Cheddar Gorge, the buildings to tempt day-trippers may be distasteful, but at Lynton, dwarfed by the surroundings, the Victorian buildings were quite pleasant. Their fate was, however, inevitable and was unfortunately sudden and tragic. Scott Simpson [4] has summed it up : " The valley form of the lower reaches is a V-shaped gorge, so narrow at the bottom that there is no room for a road beside the river. The valley-sides are too steep for cultivation and are clothed with oak-woods, with here and there a cliff of naked rock. There is no room in these valleys for a village and no call for one because the arable land lies several hundred feet above their floors. The single exception is Lynmouth which originated as a fishing village and developed as a resort. But even here, at the river mouth, there has only been room to build on the river-bed itself, as has been proved by the flood which washed away so many houses merely by claiming for itself again the boulder beds on which they had been built."

Dealing with a river carrying a heavy bed load is an unusual problem in England. Check dams have been built in Cumberland rivers to collect shingle,[7] but at Lynmouth there are large boulders as well and the velocities are very high indeed.

Following European practice,[8] recommendations were made to construct

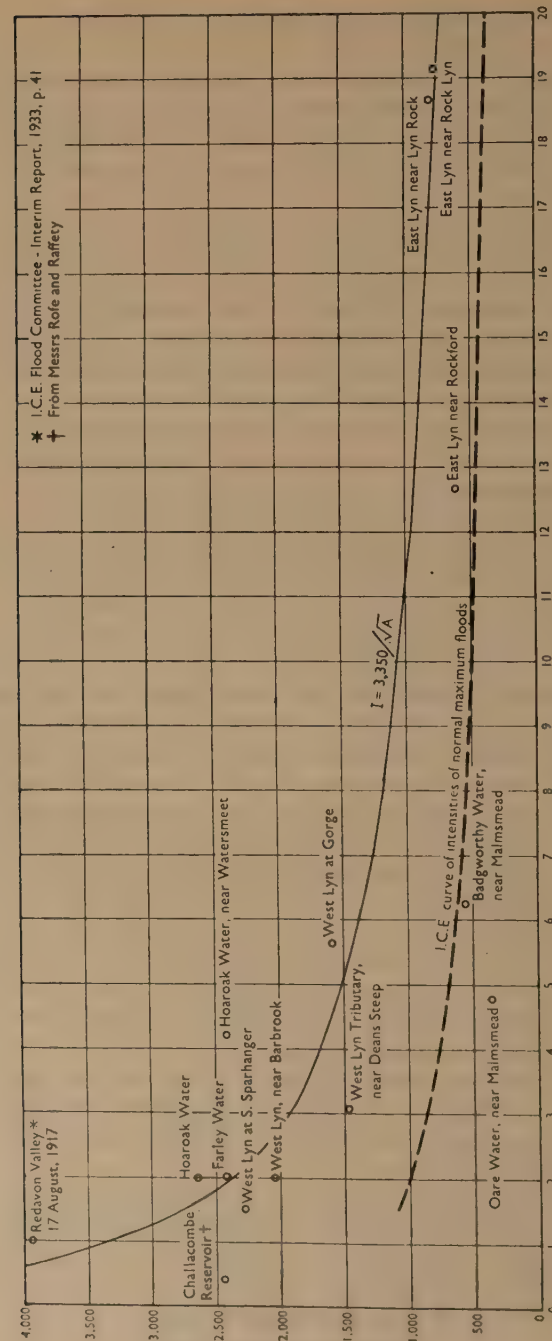


Fig. 7

FLOOD-INTENSITY/AREA DIAGRAM

Fig. 1

HEAD WATERS OF WEST LYN RIVER

Fig. 2

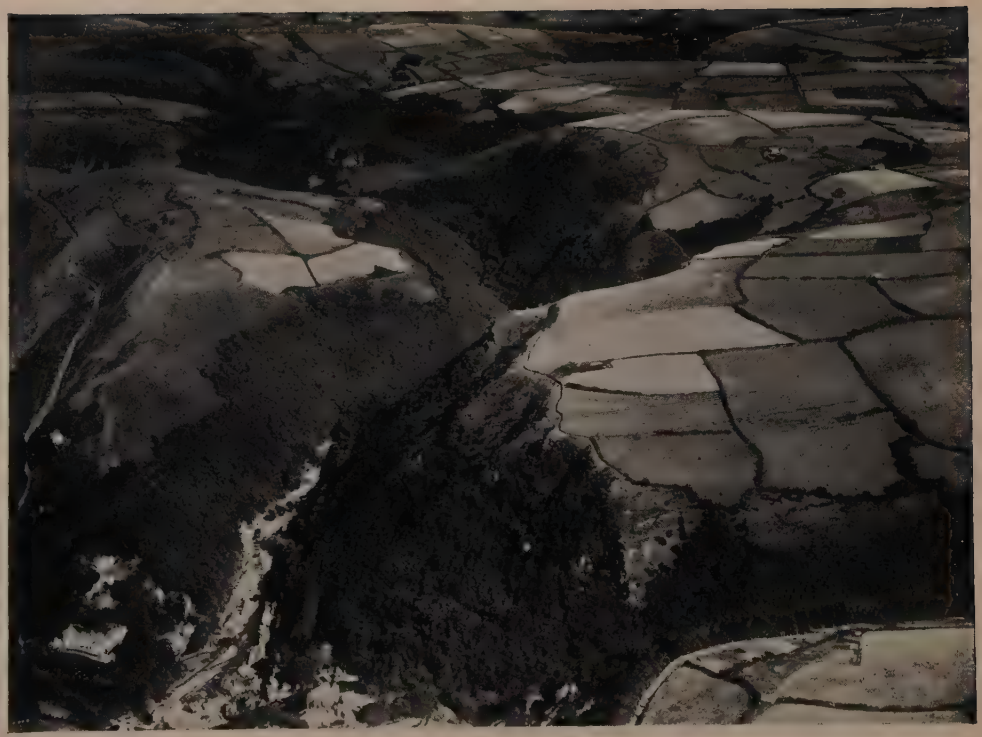
JUNCTION OF WEST LYN RIVER AND TRIBUTARY AT BARBROOK

Fig. 3

Fig. 3 shows how the lower reaches of the valleys have lost their maturity, having interlocking side spurs and, in the beds, gorges, cascades, and waterfalls.

In the bottom left-hand corner the sea-level town of Lynmouth is partly obscured by the town of Lynton, which lies in the dry valley 450 feet above.

At the left of the picture the Countisbury Hill road can be seen winding up the hill to the east from Countisbury Bridge.



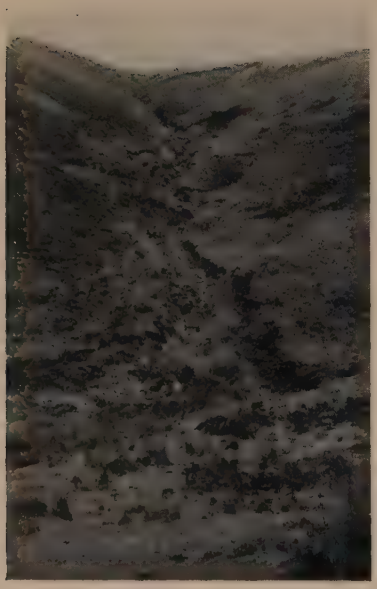
Junction of the West Lyn and East Lyn Rivers at Lynmouth

Fig. 4, on the opposite page, is a closer view of the junction of the West Lyn and East Lyn rivers, and shows their joint course to the sea. On the right are the grounds in front of Manor House.

Fig. 4

Lynmouth Five Days after the Disaster

Fig. 11



Tributary of Hoaroak Water. View looking East from Nat. Grid. Ref. 27467E 14233N, 9th November, 1952

Fig. 12

West Lyn Gauging Site. View Upstream from North Bank of Cross-Channel

Fig. 14

Scale Model under Test. Flow corresponds to Peak Flood of August 1952

through Lynmouth to the sea a channel of adequate cross-section to pass the maximum assessed flood and to ensure that this channel remained available by stabilizing its bed and sides and by collecting the river loads above in reservoirs created by check dams in the East Lyn and West Lyn rivers. The possibilities of afforestation were studied but it was concluded that under the worst conditions it would not make a material contribution to flood alleviation. It is possible that these recommendations will not be accepted, on the grounds that Lynmouth is not worth the expenditure and that the inhabitants are opposed to a large safe channel.

As many readers will sometime have been in North Devon they will recollect the features mentioned without difficulty, the hog-backed cliffs falling to the sea, the old cliffs line around the top of Exmoor, the steep bare upper reaches, the meadowed middle reaches, the deeply incised and wooded precipitous courses to Lynmouth, and the dry Valley of Rocks. The processes at work are easily appreciated by their violence and the spectacular evidence left by the past. They are at work in all river systems at all times, though not in such obvious manner and the consideration of any river problem involves their study.

Part 2.—Hydrology and Model Investigation by Mr Wolf

Hydrological Background

The complete study of the catastrophic Lynmouth flood of August 1952, entails the examination of every part of the hydrological cycle—precipitation, evaporation, infiltration, percolation and underground storage, surface storage, and surface run-off. In this case, evaporation was negligible on a day of almost incessant rain and with a layer of very damp air[9] moving over the area. A discussion of the other phases follows below.

Topography

The speed of surface flow and the rate of major surface storage largely depend on the topography of the catchment area. The general impression of the catchments of both Lyn rivers, as may be seen in *Figs 1*, *2*, and *3*, and Fig, 8, Plate 2, is one of gently rounded moors curving at ever increasing slopes down to streams which have carved deep valleys for themselves. The sides of the valleys are steep, and often precipitous, and never very far apart, so leaving little room for flood storage. The longitudinal slopes are steep, or moderately steep, as described in Part 1, and are sufficient to maintain in the streams, and in any of the small flood planes, high velocities of flow throughout their length, with velocities near the critical at the

upper ends on the edges of the moor, and in the lower stretches of the "hanging valleys."

With a generally convex topography, the time taken by rain to reach the edges of the valleys will be longer than with sharp-crested hills. The flow down the steep sides of the valleys, however, will be fast.

Geology, Moorland Vegetation, Agriculture, and Forestry.

Part 1 touches on these subjects which affect infiltration, percolation and underground storage, surface storage, and surface run-off.

The beds forming the Lyn catchment area consist of old grits, shales, and slates, of the Lower Devonian,[13, 14] covered by alluvial deposits in some parts of the valleys, and by peat or soil. The grits, shales, and slates are dense and have a very high resistance to percolation. After heavy rainfall the loss by immediate percolation may, therefore, be considered negligible.

As mentioned in Part 1, the streams have carved deep channels out of the bedrock. The debris resulting from this erosion ranges from mud and sand up to boulders weighing many tons. The flood flow set the debris in motion, and Green has calculated [10] that the total volume moved along the Lyn rivers and their tributaries exceeded half-a-million cubic yards.

The gently sloping or flat tops of the moors are in part covered in dense tough moorland grass (Gifford [11]) growing from a peaty springy ground. Other parts are covered in heather and in bracken. Burton undertook extensive observations and concluded [12] that the depth of peat or soil does not exceed 4 feet.

On the whole there are no trees on the moor tops, but there are some belts of trees in the valleys. Some of them grow in steep inaccessible places (for example, see *Fig. 12*), and there the dead trees lying near the streams were washed away by the rising waters. Other trees were undermined by the flood flow and fell, complete with their bushy root systems, into the streams. Near farms there is some arable land, and a goodly area of pasture.

The heather, bracken, and dense coarse grass of the moor, even after thorough soaking, form a dense uniform obstacle to the speedy passage of water. There was hardly any evidence, on the summits, of the vegetation being flattened by the heavy rainstorms and so providing a smoother bed for the flow of surface water towards the edge of the valleys. In the valleys, however, the streams scoured scars on each bank, in which the grass was flattened ; the bracken and heather were broken and removed, so giving a smoother surface and less resistance to flow, and in some stretches the soil was eroded.[9, 10]

Rainfall

The average-rainfall map of the Meteorological Office, published by the Ordnance Survey, shows that during an average year the rainfall at the top

of Exmoor Forest exceeds 60 inches. From there to Lynmouth the average annual rainfall decreases gradually to approximately 40 inches.

The two heaviest daily rainfalls recorded before 1952 by the Meteorological Office or its predecessors, both of over 9 inches in 24 hours, occurred at Bruton and at Cannington in Somerset. No reason has been advanced why that part of the country should be subject to such extraordinarily intense rainfalls, nor why other parts should be free from them. The occurrence of a third fall of about 9 inches in 24 hours, near the Somerset border, may perhaps stimulate further study of what, on the basis of present knowledge, seems no more than a random coincidence.

With regard to the more frequent rainstorms of smaller intensities, the view of one meteorologist is that the Lyn catchment receives them less often, and may be assumed, therefore, to be somewhat less subject to floods, than its neighbours to the south and to the east.

At present there are no daily rain gauges in operation in the Lyn catchment area, and the frequencies of rainstorms of various intensities cannot, therefore, be reliably estimated.

The Flood of 15th August, 1952

Rainfall

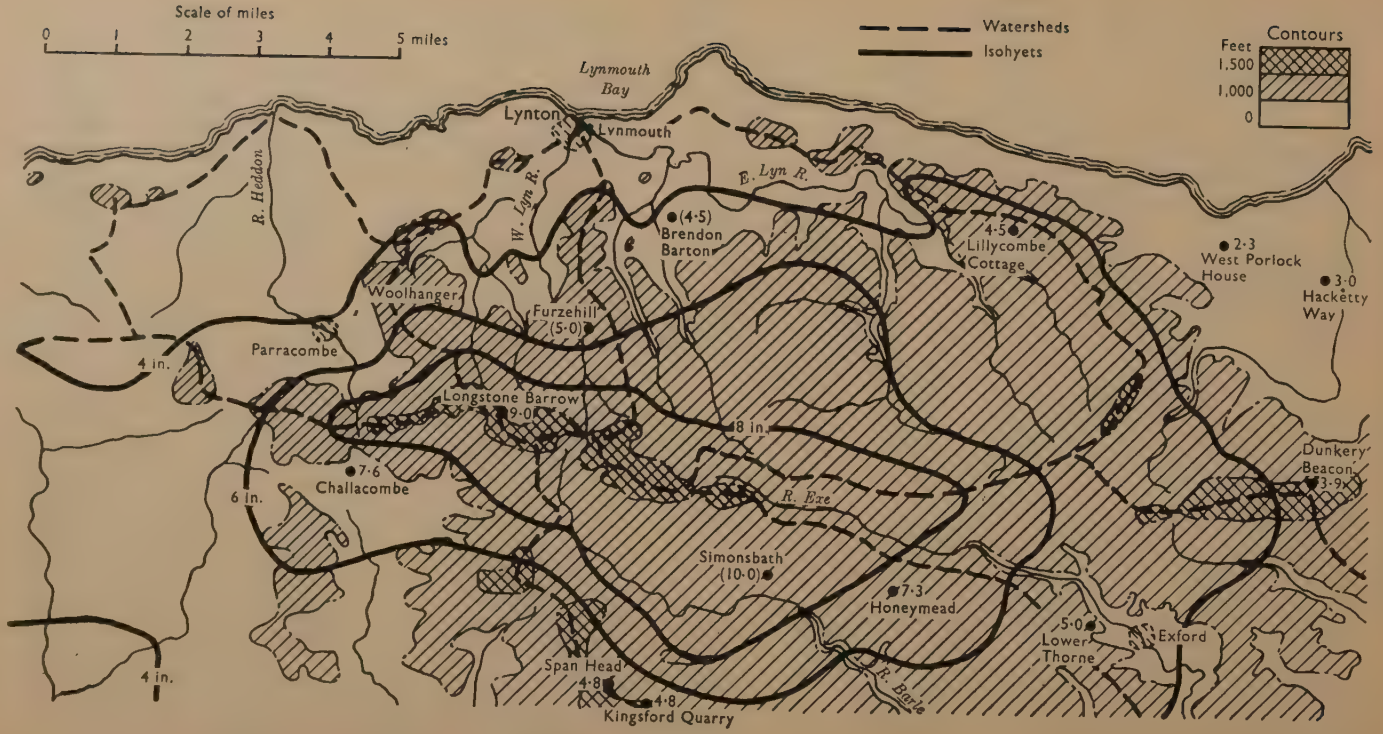
The isohyetal map derived from the Paper by Bleasdale and Douglas[9] and reproduced in *Fig. 9* shows the result of the work of the Meteorological Office. The Paper[9] contains some interesting computations, and gives the average general rainfalls on the West Lyn catchment of 9·1 square miles as 5·87 inches, on the East Lyn catchment of 30·1 square miles as 5·56 inches, and on the combined areas of 39·2 square miles above Lynmouth as 5·63 inches during the rainfall day. An area of 17 square miles is stated to have received a fall of more than 8 inches, 42·5 square miles more than 6 inches, and 140 square miles more than 4 inches in 24 hours. The rainfall distribution is shown to have consisted of a basic orographic pattern, with thunderstorm peaks superimposed on it.

Bleasdale and Douglas come to the conclusion that during the successive hours starting from 4.30 p.m. the distribution of rainfall in time, for Exmoor as a whole, was approximately 8, 8, 3, 10, 18, 18, and 10 per cent of the whole day's fall. On the West Lyn catchment of 9·1 square miles, the highest hourly rainfall spread over the whole area would, therefore, have been $5{\cdot}87 \times 0{\cdot}18 = 1{\cdot}06$ inch, with a corresponding fall of 1·00 inch during 1 hour on the East Lyn catchment of 30·1 square miles.

The accuracy of the isohyetal map cannot now be checked. It is the product of a great amount of painstaking work, and any corrections which may have to be made will clearly be within the limits of accuracy to which engineers are accustomed in this field.

To engineers who are concerned with rainfall figures primarily for their use in the determination of run-off, it may be of interest to find that the

Fig. 9



RAINFALL DISTRIBUTION OVER EXMOOR, 15TH AUGUST, 1952

(Figures in brackets are approximate readings obtained by unorthodox methods.)

rainfall contours have been plotted, in many cases, on the basis of computations (in the reverse direction) from flows required to produce observed erosion and deposition patterns. The difficulties of such computations are well brought out in the discussions of the geology and of the vegetation and soil by G. W. Green[10] and Mrs Gifford.[11] Green has drafted a map of highest rainfall, based on his very thorough geological observations, and his 8-inch contour would cross the Hoaroak valley where Bleasdale's 6-inch contour occurs.

The present Author's walking survey of erosion patterns in that area (for example, see *Fig. 11*) also suggests higher rainfall figures in the Hoaroak valley than are shown in *Fig. 9*. It is impossible to obtain a high degree of accuracy in this work, for the same amount of erosion may be due to short intense flows or to longer flows of less intensity but greater total discharge. Any local adjustment of perhaps 20 per cent would still maintain a very satisfactory overall accuracy of the map by Bleasdale and Douglas.

The only rain gauge in the Lyn catchment is at Longstone Barrow, at the head of the West Lyn, on the divide between the rivers Lyn and Bray. This is an octapent, read by the voluntary rainfall observer in this area, Mr C. H. Archer, to whom the Authors are so greatly indebted for the observations during August 1952, on which much of this work is based. Mr Archer read the gauge on the 1st August, and again at 4 p.m. on August 14th when he recorded 6·46 inches ; he estimates that a large proportion of this amount fell during the period 5th to 9th, with another heavy fall on 11th and light rain on the 12th and 13th August. After the storm causing the disastrous flood, Mr Archer recorded 9·04 inches at 12.30 p.m. on the 16th August. From other evidence the Meteorological Office concluded that 9 inches of rain fell during the 24 hours preceding 10 a.m. on the 16th August, and of this amount 7·6 inches fell between 4.30 and 11.30 p.m., and 5·9 inches between 7.30 and 11.30 p.m. The highest rate of rainfall was estimated at 1·6 inch per hour, and occurred between 8.30 and 10.30 p.m.

Gauges near the catchment are established west of Challacombe (where 7·58 inches were recorded for the 24-hour period preceding 10 a.m. on the 16th August), east of Simonsbath (where 7·35 inches fell during the same period), at Dunkery Beacon, Wootton Courtenay (approximately 4 inches at each), and near Porlock (2·3 to 3·0 inches).

The observations at the nearest automatically recording gauges, which are installed at Ford Cottage, Wootton Courtenay, to the east and at Chivenor, to the west, have been used[9] to deduce the rainfall intensities and durations over the Lyn catchment, and in particular at Longstone Barrow. *Fig. 10* contains, on a base of Greenwich Mean Time, graphs of the rates of rainfall, in inches per hour, during successive 6-minute intervals. At Wootton Courtenay, with a daily total of 4·08 inches, a fall depositing about 0·3 inch of rain in 50 minutes occurred at 2 p.m., another of 0·8 inch

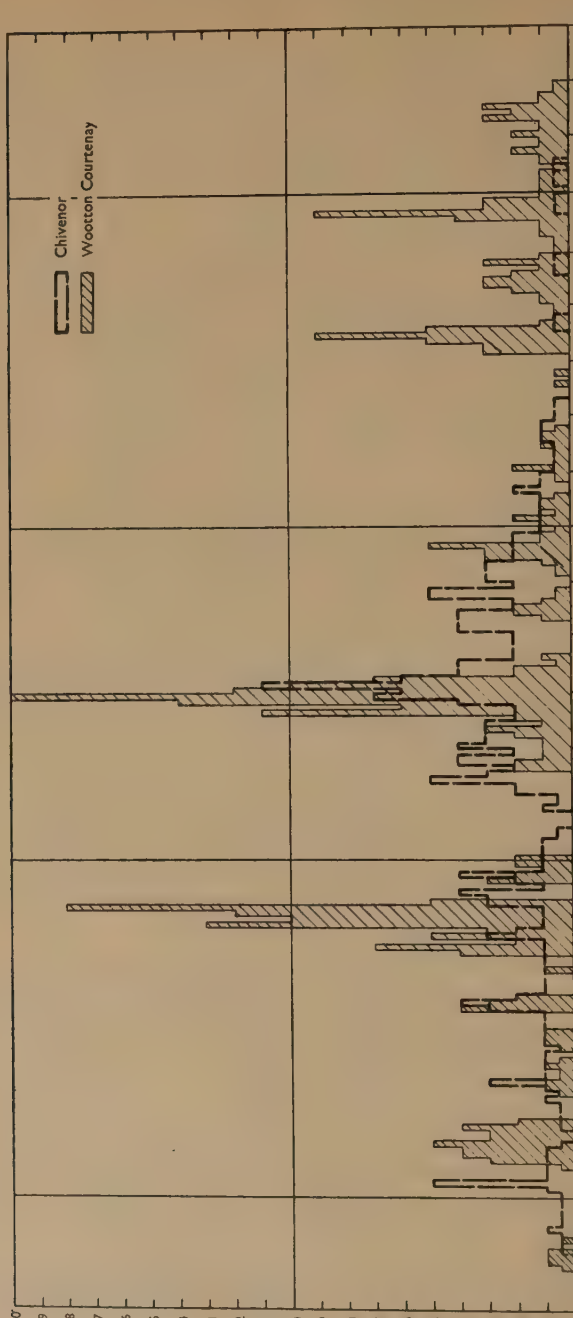


Fig. 10

RATES OF RAINFALL AT CHIVENOR AND WOOTTON COURTENAY, 15–16TH AUGUST, 1952

in about 1½ hour at 6 p.m., and a third, of about 1 inch in 2 hours, at 10 p.m. The peak intensity of precipitation appears to have been 2 inches per hour at 10 p.m., but short bursts of high intensities occurred throughout the easier rainfall during the remainder of the day. At Chivenor, with a daily total of 2·67 inches, heavy rain started at 1 p.m. and continued until 2 or 3 a.m. on the 16th August, with the heaviest falls at 6.30 and from 8.30 until midnight, and the highest intensity of precipitation of about 1·2 inch per hour, at 10 p.m.

The evidence of persons who had been at Lynton or Lynmouth, or in valleys nearby, indicated that heavy rain began to fall at or soon after noon. Some people stated that they had observed two distinct periods of heaviest rainfall, around 6.30 and 9 p.m. From a comparison with the records from Wootton Courtenay and Chivenor, such evidence has a strong claim to serious consideration.

The peak intensities during the heaviest falls on the Lyn catchment cannot be accurately determined, but they may be estimated by three different methods.

Bleasdale and Douglas[9] deduce peak rainfalls of 1·6 inch per hour at Longstone Barrow from falls of about 0·5 inch per hour at Wootton Courtenay and somewhat less at Chivenor.

Observations of scour and deposition lead to even higher local values than 1·6 inch of rain in 1 hour. On the basis of his geological observations, Green[10] suggests that rainfall at the head of the Hoaroak valley was heavier than at Longstone Barrow. The summary in Table 1 of the flood flows computed by Mr Dobbie supports Green's suggestion of high rainfall intensities at the head of the Hoaroak valley.

A further example would be the catchment area of the stream shown in *Fig. 11* (a small tributary of the upper Hoaroak Water), which is less than 20 acres; the area of waterway below the erosion scars was more than 40 square feet, and velocities compatible with the steep slope of the bed yield an estimated flow of 300 cusecs or more, equivalent to a rate of run-off exceeding 10 inches per hour, presumably maintained for the period of concentration of the small area. Confirmation (based on theoretical considerations) has been received from Mr B. J. Mason, of the Department of Meteorology at Imperial College, that rainfalls of intensities exceeding 10 inches per hour and lasting for 10 minutes or more occur on small areas (of the order of ⅛ square mile) in Great Britain.

Applying probability computations to known rainfalls of longer duration at Longstone Barrow, the heaviest local fall may be calculated to be approximately 4 inches in 1 hour.

Run-off

No gaugings of river flows during the flood are in existence, and the methods employed subsequently by Mr Dobbie's staff, to obtain the

TABLE 1

No. of gauging station	River	Site of gauging station	Catchment area : acres	Average of flows computed from site surveys : cusecs	Flows added arithmetically : cusecs	Rates of flow :		I.C.E. normal maximum flood rate of flow, cusecs 1,000 acres	Ratio of deduced rate of flow to I.C.E. normal maximum flood rate.
						cusecs per 1,000 acres	inches per hour		
1	Oare Water	Oare	4,820	1,720	—	360	0·36	710	0·5
2	Badgeworthy Water	Malmsmead	6,250	3,450	—	550	0·55	625	0·9
1 + 2	—	—	11,070	—	5,170	—	—	—	—
3	East Lyn	Rockford	12,680	7,680	—	610	0·61	460	1·3
4	Farley Water	Hillsford Bridge	2,010	4,870	—	2,420	2·42	1,010	2·4
5	Hoaroak Water	Hillsford Bridge	1,990	5,250	—	2,640	2·64	1,010	2·6
4 + 5	—	—	4,000	—	10,120	—	—	—	—
6	Hoaroak Water	Watersmeet	4,200	10,100	—	2,410	2·41	750	3·2
7	East Lyn	Lyn Rock	18,770	15,400	—	820	0·82	400	2·1
8	East Lyn	Rock Lyn	19,170	14,700	—	770	0·77	390	2·0
9	West Lyn	South Sparhanger	1,530	3,480	—	2,270	2·27	1,100	2·1
10	West Lyn	Barbrook	2,000	4,080	—	2,040	2·04	1,010	2·0
11	West Lyn Tributary	Dean Steep	3,090	4,550	—	1,470	1·47	860	1·7
10 + 11	—	—	5,090	—	8,630	—	—	—	—
12	West Lyn	Gorge	5,630	8,900	—	1,580	1·58	660	2·4
8 + 12	—	—	24,800	—	23,600	—	—	—	—
Estimate adopted	Lyn River	Lynmouth	24,960	—	23,000	920	0·92	350	2·6

survey data necessary for flood-flow calculations, are described in Part 1.

Eye-witnesses have emphasized the occurrence of frightening flood waves or surges in the Lyn rivers, which were, no doubt, caused by recurring ponding and subsequent release of flood waters from temporary reservoirs formed by trees and boulders piling up, or "jamming," and giving way. The high-water marks may, therefore, indicate discharges of short duration only, and it is doubtful whether these peak flows from the tributaries reached the main rivers simultaneously.

Table 1 contains a summary of the flows calculated by Mr Dobbie from the survey data of his assistants. Mr Dobbie has compared these flows (see *Fig. 7*) with the normal maximum flood intensities, as plotted in the Interim Report, of 1933, of the Institution Committee on Floods in Relation to Reservoir Practice, without allowance for any possible differences between the Lyn catchment and the "average" upland catchments envisaged by the Committee.

Gauging station No. 12, on the West Lyn, was close to the site subject to a check by the model. As will be seen later, the flow following the high-water line in the zigzag part of the West Lyn gorge was calculated from model readings to be 7,800 cusecs, and this value provides a check for the calculations based on survey data with a judicious use of the Manning formula.

Summary

The distribution in time and in space of the rainfall on the catchment, as computed by Bleasdale and Douglas[9] on the basis of the scant data available, yields an average value of the rainfall over the whole area of more than 5½ inches in 24 hours. The heaviest fall, averaged over the whole area, was computed to amount to about 1 inch per hour. Local peak rainfalls approaching 4 inches in 1 hour would clearly be consistent with the general picture.

A great deal of rain fell on the catchment during the fortnight preceding the flood; the rain gauge at Longstone Barrow recorded approximately 6½ inches between the 1st and 14th August.

Evaporation during the day of the flood was negligible.

The permeability of the surface is low over most of the area. The storage capacity of the vegetation, peat or soil, and rock may be assumed to have been taken up by the rain which fell before the 15th August, leaving virtually no opportunity for infiltration to occur after the first few showers on that day.

Surface storage on the high moors, with little ground slope and a dense cover of vegetation, is difficult to separate from the formation of a continuous sheet of water discussed below; but on the convex hills there would be little storage in ponds, and that capacity would be filled up soon after the commencement of rain.

Surface run-off on the high moors must have consisted of the movement of a sheet of water, impeded by the thick stalks of plants growing close together. What artificial drainage channels had been cut in that area during the nineteenth century had proved inadequate even during normal times, and after the exceptionally heavy rainfall would certainly carry only a negligible proportion of flow. The average velocity of flow over the summits may have been a few inches per second. Water had to travel hundreds, and in some cases thousands, of feet before reaching a stream or the edge of the moor, at average rates of perhaps 10 to 20 feet per minute. By the time it reached the stream or the abrupt edge, the sheet of water must have reached a thickness of several inches. The duration of its journey may have been up to an hour or two. Down the steep slopes of the valley-sides, velocities must have been several feet per second, and in the streams the average flood velocities down to the gauging sites were 5 to 10 feet per second, or even more. At the height of the flood the time of flow from the headwaters of the West Lyn down to the sea was less than 1 hour, and perhaps two or three times that figure in the various tributaries of the East Lyn.

Peak run-off rates, as summarized in Table 1, amounted to 2·64 inches per hour (Upper Hoaroak Water), 2·4 inches per hour approximately (Farley Water and Lower Hoaroak Water), 2·27 inches per hour (Upper West Lyn), and 2·04 inches per hour (Middle West Lyn).

These rates of flow, calculated from high-water marks, are not likely to have been maintained for long. The many trees and boulders moving downstream clearly piled up in places and formed small temporary reservoirs which would burst their banks and sweep on as the " walls of water " described by eye-witnesses. For short periods, run-off intensities were as high as, or even higher than, rainfall intensities.

Estimates of Maximum Floods

Estimates based on the Records of the Flood of August 1952

During the period following the flood disaster it was noticeable that occasional rainfalls of usual intensities caused a very speedy rise in river flows. This increase in the immediate run-off factor, compared with conditions before the flood, was at first attributed to the after-effects of the storm : the catchment had received so much rain that all means of retaining further rain were considered exhausted for a period of days or weeks. Subsequently it became evident that the effect was more permanent. *Fig. 11* shows an example of the increase of the discharge capacity of a stream by scour. There were clearly countless similar, but smaller, improvements in the drainage of the approaches to the streams caused by the heavy flood run-off. Whilst much of this was caused by damage to vegetation which will, in time, go back to its previous state, it is reasonable to attribute at

least equal values to the immediate run-off factor when considering possible future floods which would again cause an improvement in drainage.

From the rainfall pattern shown in *Fig. 9* the average rainfall over the whole catchment of 39·2 square miles was computed[9] to have been 5·63 inches during the rainfall day.

The worst rainfall recorded in the British Isles, at Bruton in Somerset, amounted to 9·56 inches in 24 hours. The average rainfall intensity over the 25,000 acres surrounding Bruton was 80 per cent of 9·56, or 7·65 inches. Assuming such a fall to be possible anywhere in the country, and run-off factors to be not greatly increased because of the increased rainfall, the maximum possible river flow at Lynmouth would amount to 35 per cent more than the peak flow of August 1952.

No survey data are available for the calculation of the peak flood flow, last August, in the river downstream of the Lynmouth junction. Working from rainfall intensities, the Author obtained a value of approximately 22,000 cusecs. This figure, though less reliable than the others, leads to a value for a maximum flood flow, arising from a rainstorm of Bruton intensity striking the Lyn catchment, of 30,000 cusecs.

The West Lyn catchment, with an area of 9·1 square miles, received an average rainfall of 5·87 inches in 24 hours. The average rainfall on an equivalent area surrounding Bruton was 9 inches. Such a fall would cause a discharge in the West Lyn, at Lynmouth, more than 50 per cent greater than the recorded flow. The 8,000 to 8,500 cusecs would, given a rainstorm of Bruton intensity, be increased to more than 12,000 cusecs.

On an area of the size of the East Lyn catchment, the Bruton rainfall amounted to 40 per cent more than the 5·56 inches in 24 hours computed by Bleasdale and Douglas. Mr Dobbie's figure for the peak flow on the East Lyn, 15,400 cusecs, leads to a maximum discharge for that river of nearly 22,000 cusecs.

Estimates based on I.C.E Floods Report

The Institution Committee on Floods in relation to Reservoir Practice consisted of some of the engineers most eminent and experienced in the field of flood control. They had access to extensive British records, and their conclusions and proposed methods are designed to be applied generally throughout the British Isles.

The Interim Report of 1933 of that Committee, briefly referred to as the I.C.E. Floods Report, covers particularly floods from catchments up to 25,000 acres in area, where the mean inclination of the straight line from source to mouth of the main stream is between 1 in 10 and 1 in 50.

The total catchment area upstream of Lynmouth is 24,960 acres. The mean inclination of the East Lyn (including any major eastern branch) is about 1 in 50, and that of the West Lyn about 1 in 20 . The conclusions of the I.C.E. Floods Report may, therefore, be taken as especially relevant to this case.

Normal Maximum Floods

West Lyn catchment alone . . .	5,720 acres	3,780 cusecs
East Lyn catchment alone . . .	19,240 acres	7,120 cusecs
Combined catchment	24,960 acres	8,250 cusecs

It is to be expected, on the assumptions of the I.C.E Floods Report, that the normal maximum flood of the combined catchment will be highest. This would be the result of a rainfall of about 3 inches falling in a few hours corresponding to the period of concentration of the area.

Prolonged Catastrophic Floods, in view of the small amount of flood-plain storage in the Lyn valleys, are not likely to have a peak flow much in excess of that of the normal maximum flood.

Acute Catastrophic Floods, on the other hand, will have a very high peak flow arising from rainfalls in excess of 3 inches lasting over the period of concentration.

According to the I.C.E. Floods Report's Formula for maximum rainfall intensities in the British Isles, $I = \frac{8}{T+1}$, the ratios of peak flows, when calculated from the times of concentration of the various areas according to Bransby Williams's formula, give rise to the following estimates :—

On the combined catchment, with a time of concentration of 7·15 hours (that of the East Lyn part), and an area factor of 75 per cent, the average rainfall over the whole area would be about $5\frac{1}{4}$ inches in 7·15 hours, giving a peak flow of 8,253 $(5\frac{1}{4}/3) = 14{,}500$ cusecs.

On the East Lyn catchment alone, with an area factor of 77 per cent, the peak flow would be 12,830 cusecs.

On the West Lyn catchment alone, a time of concentration of 3·1 hours and a 92-per-cent area factor would result in a peak flow of about 7,000 cusecs.

If, however, the rainfall intensities observed at Bruton are applied to the Lynmouth area, these peak flows will be increased to 19,250 cusecs for the combined catchment, 17,200 cusecs for the East Lyn catchment alone, and 8,850 cusecs for the West Lyn catchment alone.

An analysis of the velocities of flood flow in the rivers East and West Lyn (see p. 536, *ante*) indicates that the times of concentration deduced from Bransby Williams's formula are too long and should be corrected to approximately 3·5 hours for the East Lyn and 2 hours for the West Lyn. According to the maximum-rainfall formula, $I = 8/(T + 1)$, the average rainfall over the whole area of the combined catchment would be about 4·7 inches in 3·5 hours. This represents a greatly increased rate of rainfall per hour as compared with the figures quoted above, and would cause peak flows considerably greater than those quoted in the last paragraph.

Maximum Size of Boulders in Valley Alluvium

G. W. Green has confirmed, in conversation, an observation which proves that, in the past, floods have occurred which were greatly in excess of that of August 1952, although their dates, numbers, or frequencies are unknown.

The flood of last August caused erosion in the river beds, as described earlier, and exposed in the banks boulders of huge size, whose outline gave proof that they had been rolled there. Until the recent erosion they had rested under layers of smaller debris and a cover of soil and vegetation. The age of the cover was no doubt considerable, pointing to very long recurrence intervals between these heavy floods, but the size of boulders appears quite consistent with the peak flows estimated above.

Historical

When designing river-control works it is desirable to know the probable recurrence intervals of floods of various intensities. Such information is most accurately derived from systematic river-gauging records covering long periods.

The flows in the rivers East and West Lyn do not appear to have been gauged in the past.

Unfortunately the second means of assessing probable recurrence intervals of floods—historical accounts of earlier disasters—is also insufficient for the purpose. The most reliable book [16] on the history of the area mentions a flood in 1796 during which

> " the river at *Limouth* by the late rain rose to such a degree as was never known by the memory of any man now living, which brought down great rocks of several ton each. . . ."

This flood may have been similar in intensity to that of August 1952.

Other storm damage to Lynmouth mentioned in this book is assumed by the present Author to have arisen chiefly from the action of the sea rather than that of the rivers.

From early prints reproduced in the same book one reason why the loss of life in August 1952 was unprecedentedly heavy stands out very clearly : a view dated 1830 shows the river East Lyn, although confined between the abutments of a two-span arch bridge, wide and unconfined elsewhere. There were no houses and no retaining walls to throttle the flood flows in either river and to hold back trees and boulders.

Model Investigation

Part 1 and the earlier sections of Part 2 contain discussions of the need for run-off figures, both during the flood of August 1952, and in more normal times, in the rivers of the Lyn area. Without this information it

would clearly be impossible to design river improvements and associated works with any degree of confidence in the success of the scheme.

As soon as Mr Dobbie was invited to consider the problem of the safety of Lynmouth and its neighbourhood, he recognized the advantage of assessing river flows by using two independent methods, and it was he who discovered the gauging site on the West Lyn. When the present Author arrived at Lynton, shortly after, Mr Dobbie suggested that a model study be undertaken.

Description of Gauging Site

650 yards upstream (or south) of the confluence of the West Lyn and East Lyn rivers, at the head of the steepest section of the West Lyn gorge, the flow is confined between steep sides of rock forming a channel remarkably regular for a natural gorge. This channel turns, in quick succession, through two approximately right-angled corners, and from the second corner the water emerges in a series of waterfalls into the West Lyn valley just above Lynmouth. Boulders reaching the approach channel are moved through this stretch at high speeds, and although the receding flood left a continuous layer of boulders behind, in the approach channel, there were few boulders left at the corners or in the cross-channel between. The roles played by the various parts of the gorge will be described later, but it is of importance that the walls and floor of the critical section consist of solid rock which would erode, and so change the characteristics of the gauging site, too slowly to call for frequent re-calibration.

Fig. 12 indicates the abundant vegetation in the damp, sheltered valley, above the rocky channel. The flood of August 1952 left behind it a high-water mark consisting in the approach channel (as further upstream) mostly of plant debris, presumably washed up by the peak flow or by waves, and left behind as the flood receded. It is possible that later rainfall caused some of the grass and twigs so left behind to be washed down the slope, but no evidence of this was found. Little sign of plant debris could be found on the steep rock walls of the cross channel. There the highest flood had reached up into the soil precariously resting on the rock at higher levels, and washed it away up to a fairly clear line.

Survey of Gauging Site

The site reproduced by the model consists of a 350-foot length of the approach channel, of the cross-channel, and of the first two waterfalls downstream

Mr Dobbie's survey party obtained levels and cross-sections of the channel. Alignments and details were obtained by the present Author by photo-theodolite and ordinary camera.

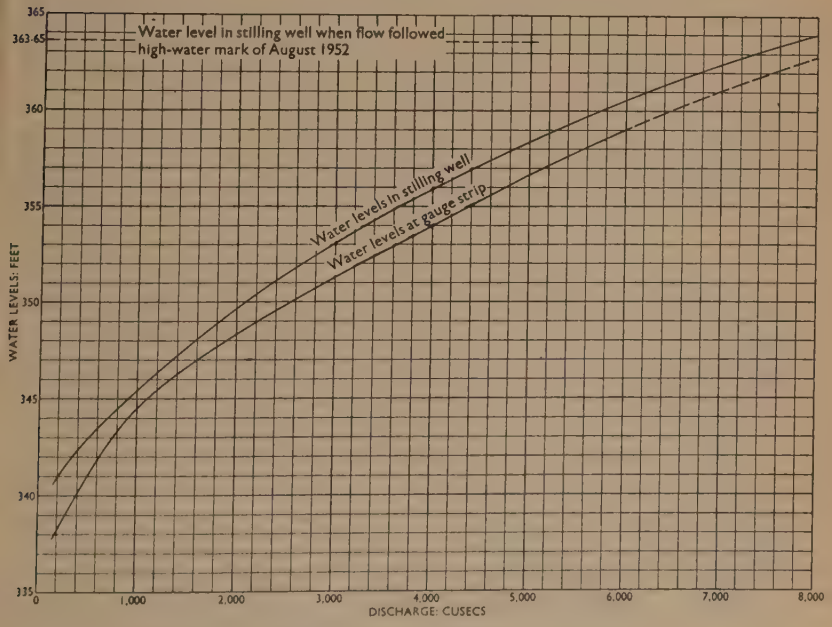
Theory and Purpose of Model

The other gauging sections in the area, as described in Part 1, consist of straight channels of uniform cross-sections and slopes. The flows through

those channels were calculated by application of Manning's formula, which required an estimate to be made of the appropriate friction coefficient. Manning's formula is one of a group applicable to flow which is governed by the roughness of the boundary. Mention is made in Part 1 of the hedges crossing the meadows adjacent to the streams. Many of these hedges grow out of ridges or low walls which, instead of behaving like bed roughnesses, act rather in the manner of weirs and, therefore, are not capable, in a strict analysis, of definition by a frictional coefficient.

At the West Lyn gauging site subject to the scale-model test, the flow is determined by wall forces and gravity effects, with frictional losses playing an insignificant role both in the sharp corners and at the terminal waterfall. From the laws of hydraulic similarity it follows that, within proper limits, a geometrically similar model constructed to a reduced scale will accurately reproduce all the properties of the flow through the natural gauging site. Conversely, a knowledge of the behaviour of the model will make it possible to predict the discharge through the natural gauging site with a higher certainty of precision than the application of frictional flow formulae to the uniform straight channel sections elsewhere permits. The stage-discharge curve derived from the calibration of the model, and shown in *Fig. 13*, will enable the site to be used as a permanent gauging section if

Fig. 13



West Lyn Gauging Site. Stage-Discharge Curves

required. By substitution of the observed level of the high-water mark in the stage-discharge curve it is also possible to determine the peak flood intensity during the night of the 15–16th August, 1952.

Arrangement of model

The scale of the model was chosen as 1 : 48 (4 feet to the inch). This was large enough to avoid appreciable scale effects at low flows, yet convenient for testing in the Hawksley Hydraulic Laboratory of the Imperial College of Science and Technology.

The channel was shaped in paraffin (model) wax and supported by a timber casing on trestles. Mr Dobbie's staff provided the casing and fashioned the outlines of the channel, and the model was then set up at the College, aligned, corrected for detail, and provided with the necessary instruments and measuring devices, as shown in *Fig. 14.*

The water supply, from a constant level overhead tank, may be metered either by a vertical water-manometer connected across a calibrated bend in the feed pipe (large flows), or volumetrically by diversion of the flow into a 15-cubic-foot tank at the discharge end (medium and small flows).

The discharge scale of the model is $1 : 48^{2 \cdot 5}$ or 1 : 15,962 which is nearly 1 : 16,000. A flood flow of 8,000 cusecs in nature is represented by $\frac{1}{2}$ cusec in the model. This is roughly the flow shown in *Fig. 14.*

Because of the steepness of the valley and the comparative inaccessibility of the gauging site, it was considered convenient to establish the gauge in a position where it could be read through binoculars from the main road higher up. Mr Dobbie, therefore, arranged for a scale to be painted on the rock of the north wall of the cross-channel, a short distance from the upstream corner. This has been shown by the model to be a suitable section for a gauge. It is, however, difficult to determine the exact level on the gauge, by eye, at high discharges when the surface of the water is much disturbed and the flow surges up and down the gauge over a range of several feet.

Should this site, which is undoubtedly hydraulically the most suitable one on the lower West Lyn, be chosen as a permanent gauging station, it would be found advantageous to install equipment including a device for damping the surges against the gauge, such as a stilling well commonly used with a float recorder. A stilling well was in fact installed on the model, connected to an opening at the foot of the north wall of the cross channel, exactly opposite the spur of rock formed by the approach channel and the cross channel. Its top may be discerned at the right of *Fig. 14.* A micrometer point gauge could be read to 0·001 inch in the stilling well, corresponding to 1/20th inch in nature. The accuracy of observing the water surface against the gauge strip on the rock, however, was poor, and at heavy flows successive readings taken at the same discharge varied by amounts representing several inches in nature. *Fig. 13* shows that 1 foot of rise, at high flows, represents an increase in discharge of approximately 500 cusecs.

Results

The stage-discharge curve obtained from the model is shown in *Fig. 13.*

Experiments were conducted, at medium and heavy discharges, to discover the part of the channel which governed the pattern of flow at the gauging strip and at the opening to the stilling well. The approach channel narrows down at the first corner, and it was found that little or no change in flow conditions downstream of the first corner resulted from any interference with the bed or sides of the approach channel above the convergent end. Obstacles at the first corner and on the bed and the north wall of the cross-channel, near the first corner, caused major disturbances, but obstacles on the south wall of the cross-channel or some distance downstream of the gauging strip again did not matter greatly. The conclusion from this observation is that interference with gauging records, if instituted, would arise only from boulders close to the gauge.

It is, moreover, clear that the model includes more than the minimum length of approach required for true similarity.

Observations of the effect of obstructions, at low flows, may be conveniently made at the site.

A theoretical investigation of the flow at corners in channels, and of their suitability for flow measurement, has been started by a group of senior undergraduates at the Imperial College. It is hoped to present their conclusions later.

Acknowledgements

The investigation on which Part 2 of the Paper is based was started, at Mr Dobbie's request, shortly after the Lynmouth flood disaster, and parts of it are continuing. In view of its associations, the work has received generous encouragement from many directions.

Acknowledgement is due to the Imperial College Authorities, and particularly to Professor A. J. S. Pippard, M.B.E., D.Sc., M.I.C.E., and Professor C. M. White, D.Sc.(Eng.), Ph.D., for permitting the model study to be undertaken in the Hawksley Hydraulic Laboratory. The Author is especially indebted to Professor White for his advice and criticism, to Dr J. Glasspoole, Ph.D., M.Sc., of the Meteorological Office, for the information on rainfall, to his colleagues Mr D. H. Kent, M.A., Stud.I.C.E., for his assistance in the field, Mr T. O'Donnell, M.A., for his ciné-photography, and to Mr W. A. V. Bryan, J.P., and Mr F. H. Crawley for their help with the model.

Figs 1, *2*, *3*, and *4* are reproduced by permission of the Air Ministry, and *Figs 9* and *10* are reproduced from the *Meteorological Magazine*, vol. 81, pp. 354 and 357 (December 1952), by permission of the Controller of H.M. Stationery Office.

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The Paper is accompanied by eight photographs and five drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

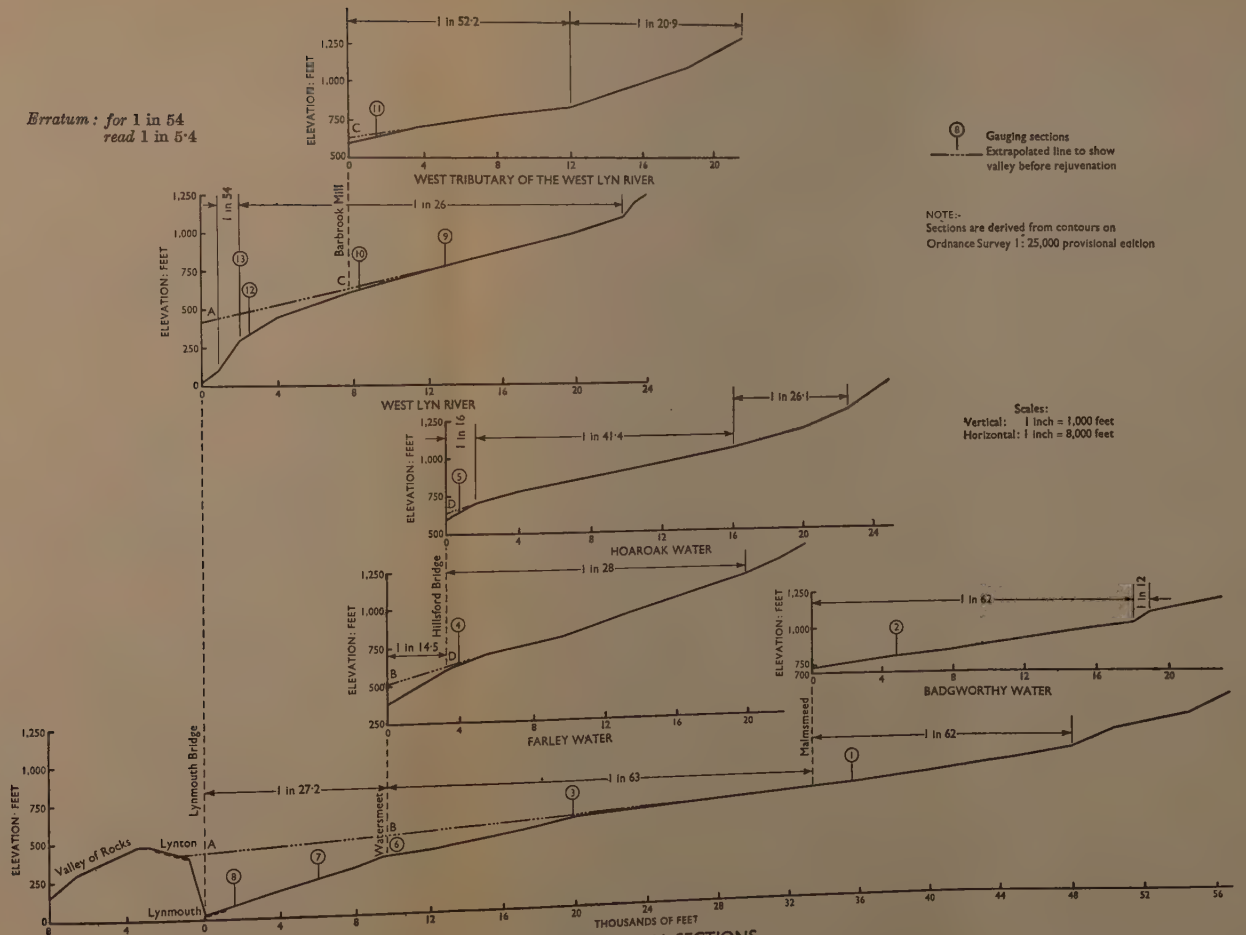
Discussion

Mr Dobbie, in introducing the Paper, said that it was not inappropriate that at the first meeting of the Hydraulics Division, in which it was hoped that there would be many fierce controversies on the finer points of hydraulics, stress should be laid on the setting of the hydraulics in the case under discussion—that was, the geomorphology of the site, which in the case of Lynmouth was all-important. At all stages in the calculations that had to be borne in mind, because there could be no understanding unless

THE LYNMOUTH FLOOD OF AUGUST 1952

FIGS 5

Erratum: for 1 in 54
read 1 in 5·4



LONGITUDINAL SECTIONS

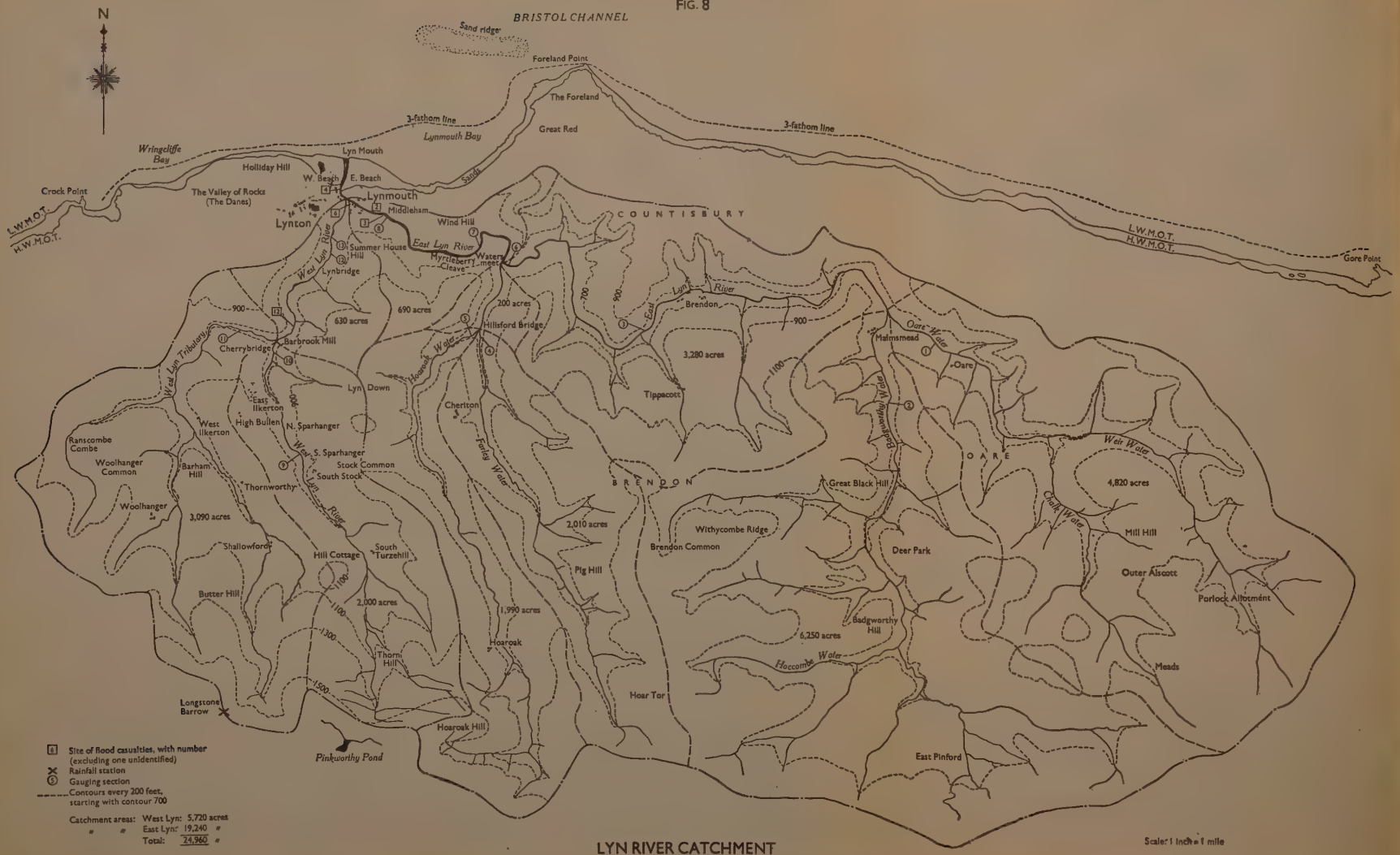
WILLIAM CLOWES & SONS, LIMITED: LONDON

The Institution of Civil Engineers Proceedings, Part III, December 1953

C. H. DOBBIE and P. O. WOLF

THE LYNMOUTH FLOOD OF AUGUST 1952

FIG. 8



LYN RIVER CATCHMENT

C. H. DOBBIE and P. O. WOLF

The Institution of Civil Engineers Proceedings, Part III, December 1953

WILLIAM CLOWES & SONS, LIMITED: LONDON

one could think in terms of the physical background of the river. It was in the Devonian rock which had been bedded down and then squeezed, so that there were cleavage planes at right angles to the bedding planes, and therefore boulders were formed very easily.

In response to requests from several members, Mr Dobbie displayed a slide (reproduced as Table 2) giving further information on the gauging stations, including the values of n adopted.

Acknowledgement was made to three people not mentioned in the Paper: Mr Harris, who was Mr Dobbie's assistant; Dr Scott Simpson, who had helped very much with the geomorphology; and Mr Chacinski, who had given great help with the hydraulics.

Mr Wolf introduced Part II of the Paper with the aid of lantern slides, and showed a film of the hydraulic scale model. In his summary of the hydrological section he laid special stress on the steepness of all valleys, as shown in Figs 5, Plate 1. Whilst the term " maturity " was appropriate in a discussion of the geomorphology of the area, to the hydraulic engineer a stream whose slope was never flatter than 1 in 75 was definitely a highland water course.

He wished to convey his and Mr Dobbie's appreciation of the work done and the help given by Dr Glasspoole's organization more emphatically than he had done in his discussion of *Fig. 9*.

Since the application of the theory of probability to problems of peak rainfall and flood had, in many past cases, been found to prove enlightening, a further study had been made of flood intensities in relation to area. If run-off were considered to be the result of many factors occurring entirely at random, an equation $I = \frac{3{,}350}{\sqrt{A}}$ could be derived, whose curve he had superimposed on *Fig. 7*. That curve showed, as a first approximation, the trend of flood intensities throughout the range of the Lyn catchments. For large areas the ordinates of the curve were approximately twice those of the I.C.E. curve of intensities of normal maximum floods, but for areas smaller than 1 acre, the ratio became much more than 2 : 1.

When the investigation described in the Paper had been carried out, the only rain gauge in the Lyn catchment area known to him was Mr Archer's gauge at Longstone Barrow, on the south-western watershed. *Fig. 9* showed another gauge, at Lillycombe Cottage, on the north-eastern watershed; though he (Mr Wolf) had not made any direct use of its rainfall record it had been taken into account by Bleasdale and Douglas,[9] whose final figures he had used in the Paper.

With regard to the scale model of the West Lyn gauging site, he added some figures on the possible errors, which might partly account for the discrepancy between the flood discharges as computed for gauging station No. 12 (8,900 cusecs) and as derived from the model (7,800 cusecs). Whilst the accuracy of water-level observation in the stilling well in the model corresponded to $\frac{1}{30}$ inch in nature (or ± 2 cusecs), the flow had to be set to

TABLE 2

No. of gauging station	River	Site of gauging station	Catchment area : acres	Cross-section No.	Area of cross-section square feet A	H.M.R.: feet R	Slope i	Coefficient n	Velocity : feet per sec. V	Flow : cusecs $Q = A \cdot V$	Average flow : cusecs
1	Oare Water . .	Oare	4,820	160	166	4·5	0·0175	0·045	11·85	1,960	1,720
				386	183	4·9	0·0062	0·045	7·6	1,380 } 1,490	
				—	50	1·32	0·0062	0·06	2·3	110 }	
2	Badgeworthy Water	Malmsmead	6,250	300	289	4·2	0·0097	0·04	9·52	2,750	3,450
				438	364	5·45	0·0097	0·04	11·34	4,130	
3	East Lyn . .	Rockford	12,680	470	616	7·6	0·011	0·045	13·6	8,350	7,680
				200	688	7·9	0·006	0·045	10·4	7,000	
4	Farley Water .	Hillsford Bridge	2,010	2	653	5·46	0·006	0·045	7·96	5,200	4,870
				3	352	4·9	0·0186	0·045	12·9	4,540	
5	Hoaroak Water .	„ „	1,990	1	426	5·35	0·0554	0·08	13·4	5,700	5,250
				2	335	5·6	0·060	0·08	14·35	4,800	
6	Farley and Hoaroak	Watersmeet	4,200	2	522	8·4	0·054	0·07	20·35	10,600	10,100
				1	431	7·44	0·075	0·07	22·3	9,600	
7	East Lyn . .	Lyn Rock	18,770	1	593	8·0	0·017	0·029	26·6	15,800	15,400
				2	772	8·12	0·020	0·0435	20·9	15,000	
8	East Lyn . .	Woodside	19,170	0	1,116	7·66	0·021	0·057	14·8	16,520	14,700
				385	841	5·54	0·035	0·057	15·28	12,880	

9	West Lyn . .	South Sparhanger	1,530	1	142	5·1	0·0305	0·045	17·1	2,420	3,270	3,480
					129	1·92	0·0305	0·06	6·6	850		
				2	225	4·17	0·0305	0·045	14·9	3,350	3,720	
					76	1·2	0·0305	0·06	4·9	370		
10	West Lyn . .	Barbrook	2,000	2	284	3·84	0·0284	0·04	15·8	4,520	5,040	4,080
					98	1·44	0·0284	0·06	5·9	520		
				1	140	4·0	0·0324	0·04	16·9	2,380	3,130	
					83	1·5	0·0324	0·06	9·1	750		
11	West Lyn Tributary	Dean Steep	3,090	1	355	5·8	0·0405	0·06	16·1	5,700	5,990	4,550
					58	1·0	0·0405	0·06	5·0	290		
				3	206	4·1	0·03	0·06	10·9	2,240	4,090	
					240	2·4	0·03	0·06	7·7	1,850		
				2	263	3·76	0·043	0·06	12·4	3,270	3,580	
					70	1·0	0·0318	0·06	4·4	310		
12	West Lyn . .	Gorge	5,630	336	610	8·5	0·0352	0·07	16·5		10,100	8,900
				200	651	8·1	0·0257	0·07	13·8		9,000	
				504	488	7·9	0·0356	0·07	15·6		7,600	

flood-marks on the walls of the gorge, where errors of, say, ± 6 inches (± 250 cusecs) might occur. There was also the difficulty of the accurate representation in a model of a natural site ; although great care had been taken, errors in the survey and in the construction of the model to the survey were unavoidable. It was hoped that the resulting error in discharge measurement was within ± 100 cusecs. The error involved in measuring heavy flows in the model was negligible.

Another source of discrepancy might arise from the difference in the causes of the flood marks. The mark at gauging station No. 12 consisted of debris washed up by what might have been a passing flood wave, whereas the flood mark at the gauging site checked by the model was an erosion scar, presumably caused by a flow of greater duration than a passing peak wave.

The Chairman said he assumed that in the model the water had not been aerated. Had the possibility of the water being highly aerated in the river been taken into account ?

Mr W. P. Thompson expressed the hope that the discussion of the results of the serious and unfortunate occurrence at Lynmouth might eventually give some guidance to people who had to live in circumstances similar to those which obtained there, and to those who still had to remain in Lynmouth. It appeared that people had gone to the place, where there was a river with an extraordinarily steep gradient (which had to be seen to be appreciated), and had built on the edge of the river very close to the bank and had in fact even encroached upon the river. They could not do that with impunity ; they would have had to pay for it at some time or another.

It was unfortunate that the ordinary person saw that the buildings were there, knew that those buildings had existed for many years and continued to exist, but could not appreciate the danger. The trained person could appreciate it, and should warn the people who could not do so.

It would be an error to consider that such an occurrence could not be repeated. It was found from rainfall statistics, that, after long intervals, there was an extraordinary precipitation which made a tremendous peak in the rainfall curve ; and then again there would be a very long period of ordinary rainfall, after which people were inclined to neglect the peaks. The arithmetical average itself, after many years, would neglect those peaks which had occurred many years back. There was thus room for forgetfulness and self-deception in those matters.

On p. 525 it was stated : " The last sizeable movement was in reverse, a rise of sea-level of 75 to 150 feet in post-glacial times, drowning many river mouths and leaving their buried outfall channels in the sea bed." On p. 526, it was stated : " The last capture at Lynmouth, which is the dominant feature of the present drainage, took place when the sea was not more than 100 feet above its present level." In one case the Authors

had referred to a rise, and in the other case to a fall. Which had actually occurred ?

Mr Thompson said that the adoption of Manning's formula, or any similar formula, was not appropriate and led to error. Such formulae had been derived from observations on the uniform flow of water. An inspection of the West Lyn stream and a guess at the conditions prevailing in the storm led to the conclusion that the flow had not been uniform. The flow would have appeared as a succession of drowned falls, and in lengths between falls there would have been severe wave action. That was mentioned on p. 537.

With regard to maximum discharge, he observed that application of the formula on p. 16 of the I.C.E. Flood Report (namely $\frac{750}{\sqrt[3]{M}}$ for the maximum intensity per square mile, and $750M^{\frac{3}{4}}$ for the maximum discharge from a catchment of M square miles) gave the value $750 \times 9{\cdot}1^{\frac{3}{4}} = 3{,}270$ cusecs.

The evidence of the automatic rain-gauges showed that, at Wootton Courtenay, 2·05 inches out of a total 4·08, and at Chivenor, 1·77 inch out of 2·67, had been recorded between 17·30 and 22·30 hours—the period of intense precipitation. (Those figures had been obtained from the Paper by Bleasdale and Douglas.) The evidence of the spread of the storm, considered in relation to the fall registered at Longstone Barrow, and coupled with the fact that the 8-inch rainfall contour covered only a part of the catchment area, showed in his opinion that the precipitation during those 7 hours had not been such as to justify the discharges shown in Table 1 of the Paper, where the rate of flow was given as double (or nearly) the maximum flood rate of the I.C.E. Report.

His own suggestion for dealing with that stream or with any streams that were dangerous was that there should be more careful maintenance of the streams and more careful design of the approaches to bridges. The bridges did not function with the maximum efficiency, and they should have approaches designed for the purpose.

Mr E. S. Crump referred to the difficulty or uncertainty in assigning a value of the coefficient n in order to calculate discharge from the observations made at site.

The old textbooks had generally been satisfied with a range of n of about 0·010 to about 0·050. He had noticed that in a recent handbook by Woodward and Posey [17] there was given a range of n from 0·009 up to 0·150—the latter, it was explained, for natural streams with extremely bad alignment and deep pools, or for floodways, with a heavy stand of timber and under-brush.

With regard to the lower limit, he remarked that, when presenting a

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[17] "Hydraulics of Steady Flow in Open Channels." Chapman & Hall, London.

Paper [18] to the Institution a year previously, he had been called to account for having used a coefficient value of n of 0·009 for a very smooth perspex pipe (which used to be included in the category of glass, brass, or earthenware, etc.) and made to admit that the 0·009, although he had thought he was quite bold in reducing it from 0·010, should really have been 0·0075, to bring it into line with modern research. Values of n could be as low as 0·0075, so that from 0·0075 to 0·150 the ratio of extremes was 1 to 20; and since velocity and discharge, with other things fixed, were directly proportional to the reciprocal of n, it meant that in assessing the value of n there was a vast range of discharge from 1 unit to 20.

He did not in the least envy Mr Chacinski the task of assigning a value to the data that he had collected from his cross-sections and slope observations to fix the values of A, R, and S. Since reading Mr Gerald Lacey's Paper some years ago, Mr Crump had often thought that if he were confronted with the task of assigning a value to n in the Manning formula, he would dodge the responsibility by having recourse to Mr Lacey's wonderful family formula, arrived at by eliminating the friction or roughness parameter f, which was really an alternative to the parameter n. By eliminating the friction parameter from two independent relationships, Mr Lacey had derived a remarkable formula which did not contain the friction factor at all and which was very simply stated as:

$$V \text{ (the velocity)} = 16\ (R^{\frac{1}{2}}S^{\frac{1}{2}})$$

So that, if it was considered legitimate to assume that the Authors were studying a number of alluvial channels, it appeared to be quite legitimate for them to be accepted into the Lacey family. All that was necessary was that each channel should fulfil the condition that it was flowing in its own detritus.

Mr Crump assumed that the Lyn system came within that category, and wherever one was concerned with assessing the discharge of an alluvial river, it appeared quite legitimate to introduce that formula and so dodge altogether the responsibility of assigning a value to n.

Incidentally, if it was required to express the discharge or velocity in terms of the Manning formula, the Lacey family formula and the Manning formula both showed V as proportional to $R^{\frac{1}{2}}$, so that by eliminating $\frac{V}{R^{\frac{1}{2}}}$, which was common to both, one was left with a very simple relationship between n and S, the relationship being $n = 0{\cdot}0929\ S^{\frac{1}{4}}$. As an example of the result that it gave, if one took the slope S as 1 in 64, $S^{\frac{1}{4}}$ was one half, and the value of n derived from the Lacey relationship would be 0·046, which certainly occurred fairly commonly in Mr Chacinski's assessments.

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[18] "Model-Tests on the Bellmouth-Spillway and Outfall of the Fassideri Project." Proc. Instn Civ. Engrs, Pt. II, vol. 1, p. 573 (Oct. 1952). See *Discussion*, pp. 602 and 609.

He had thought it worth while to work out (from an advance copy of the data which he had received from Mr Dobbie) the ratio of the values of V as obtained by Mr Chacinski and V_L as given by the Lacey formula. The results were a little disappointing; but, without wishing in the least to disparage Mr Chacinski's results, revealed with great honesty in his calculations, Mr Crump had noticed that in many cases alternative assessments of discharge differed in the ratio of about 100 : 60. For channels of the East Lyn system, the comparative values of V and V_L had rather a large range of from 72 per cent to 170 per cent, but had an average difference of only 92 per cent.

With further reference to the Lacey formula, Mr Crump observed that that formula was supported by data collected from 188 channels, of which only 17 had slopes steeper than 1 in 1,000 and only one—the Engstlugen river, with a slope of 1 in 45—had a slope greater than 1 in 100. With that single exception, to apply the formula to channels of the Lyn system was, therefore, to extrapolate it beyond the range of supporting data. Moreover, the formula contemplated a state of regime, and was therefore strictly applicable only to the dominant stage of discharge. It was therefore somewhat surprising to find it giving results not widely different from those obtained from the Manning formula.

The uncertainty of applying Manning's formula to open channels of steep slope lay in the fact that, in such channels, wastage of energy was effected in ways not contemplated by the formula, that was to say, not merely in overcoming the resistance offered by roughness of the bed and sides as manifested by turbulence having its origin at those boundaries. In steep open channels—as contrasted with closed pipes running full bore, or with open channels of flat gradient—energy wastage was also effected (*a*) by moving solid material, and (*b*) by the entrainment and emulsification of air at the surface of the stream. The latter feature had been much in evidence in the films exhibited by the Authors, which had shown that the surface water of streams in medium stages of flow was " white " water heavily charged with emulsified air. Nature's mechanism for effecting such wastage of energy was the standing-wave formed by the rapid alternation of local velocities from super- to sub-critical, the general effect being to produce a riotous medley of surface-waves. The roar of a torrent in flood was a manifestation of that action, which probably accounted for the major portion of the available fall. In such cases, it seemed reasonable to suggest that wrack-marks left by a flood would record the surface levels attained by the sub-critical buffers of the standing-waves; so that wrack-lines drawn on cross-sections would indicate a rather greater depth, and rather less velocity, than " critical."

In an irregular cross-section, the critical velocity V_C was given by:

$$V_C = \sqrt{\frac{gA}{W}}$$

TABLE 3.—CHANNELS OF EAST LYN SYSTEM

Site	Catchment area : acres	A	R	$R^{\frac{2}{3}}$	P	S	$S^{\frac{1}{2}}$	$S^{\frac{1}{3}}$	N	V_M	V_L	V_B	Q_M	Q_L	Q_B
1	4,820	166	4·50	2·726		0·0175	0·1323	0·2600	0·045	11·82	11·35	12·04	1,962	1,884	2,000
		183	4·90	2·886	37·3	0·0062	0·0787	0·1837	0·045	7·50*	8·485	12·57	1,372	1,552	2,300*
		50	1·32	1·204	37·9	0·0062	0·0787	0·1837	0·060	2·346*	3·540	6·52	117 } 1,489	177 } 1,729	326* } 2,626*
		233	3·098	2·124	75·2	0·0062		0·1837		6·39*	6·250	10·00	1,489	1,456	2,330*
												Average flow	1,725	1,670	2,165
2	6,250	289	4·20	2·60		0·0097	0·0985	0·2131	0·040	9·51	8·87	11·63	2,750	2,563	3,360
		364	5·45	3·098		0·0097	0·0985	0·2131	0·040	11·32	10·56	13·26	4,120	3,841	4,820
												Average flow	3,435	3,202	4,090
3	12,680	616	7·60	3·865		0·0110	0·1049	0·2224	0·045	13·38	13·73	15·66	8,240	8,460	9,635
		688	7·90	3·965		0·0060	0·0775	0·1817	0·045	10·12*	11·53	15·96	6,960	7,925	10·980*
												Average flow	7,600	8,192	10·350
4	2,010	653	5·46	3·102		0·0060	0·0775	0·1817	0·045	7·93*	9·01	13·26	5,170	5,880	8,650*
		352	4·90	2·885		0·0186	0·1364	0·2650	0·045	12·98	12·22	12·57	4,565	4,300	4,420
												Average flow	4,867	5,090	6,535
5	1,990	426	5·35	3·060		0·0554	0·2354	0·3182	0·080	13·37	18·65	13·13	5,690	7,940	5,590
		335	5·60	3·153		0·0600	0·2458	0·3915	0·080	14·39	19·73	13·43	4,810	6,600	4,490
												Average flow	5,250	7,270	5,040
6	4,200	522	8·40	4·135		0·0540	0·2324	0·3780	0·070	20·40	25·00	16·45	10,650	13,050	8,580
		431	7·44	3·813		0·0750	0·2739	0·4217	0·070	22·14	25·74	15·49	9,540	11,080	6,670
												Average flow	10,095	12,065	7,625
7	18,770	593	8·00	4·000		0·0170	0·1304	0·2571	0·0290	26·72†	16·45	16·06	15,850	9,750	9,510
		772	8·12	4·040		0·0200	0·1414	0·2714	0·0435	19·51	17·51	16·17	15,070	13,550	12,550
												Average flow	15,460	11,650	11,005
8	19,170	1,116	7·66	3·884		0·0210	0·1449	0·2759	0·0570	14·66	17·13	15·70	16,360	19,130	17,530
		841	5·54	3·13		0·0350	0·1871	0·3271	0·0570	15·27	16·37	13·37	12,830	13,770	11,230
												Average flow	14,595	16,450	14,380

* With flat slope, velocities are sub-critical, rendering the Boussinesq criterion invalid.

† Velocity probably hypercritical.

Site	Catchment area : acres	A	R	$R^{\frac{2}{3}}$	P	S	$S^{\frac{1}{2}}$	$S^{\frac{1}{3}}$	N	V_M	V_L	V_B	Q_M	Q_L	Q_B
9	1,530	142	5·10	2·965	27·8	0·0305	0·1746	0·3124	0·045	17·08†	14·81	12·82	24,24	2,103	1,819
		129	1·92	1·545	67·1	0·0305	0·1746	0·3124	0·060	6·68	7·725	7·86	861 } 3,285	996 } 3,099	1,013 } 2,832
		271	2·855	2·012	94·9	0·0305		0·3124			10·06	9·60		2,724	2,600
		255	4·17	2·590	53·9	0·0305	0·1746	0·3124	0·045	14·92†	12·95	11·60	3,358	2,911	2,606
		76	1·20	1·129	63·3	0·0305	0·1746	0·3124	0·060	4·885	5·65	6·215	371 } 3,729	429 } 3,340	472 } 3,078
		301	2·57	1·876	117·2	0·0305	0·1746	0·3124			9·38	9·100		2,824	2,740
												Average flow	3,507	2,774	2,670
10	2,000	284	3·84	2·450	73·9	0·0284	0·1685	0·3051	0·040	15·33†	11·96	11·12	4,350	3,398	3,160
		98	1·44	1·275	68·1	0·0284	0·1685	0·3051	0·060	5·32	6·225	6·81	521 } 4,871	610 } 4,008	667 } 3,827
		382	2·691	1·934	142·0	0·0284		0·3051			9·45	9·31		3,610	3,560
		140	4·00	2·519	35·0	0·0324	0·1800	0·3188	0·040	16·83†	12·85	11·36	2,356	1,800	1,590
		83	1·50	1·310	55·3	0·0324	0·1800	0·3188	0·060	5·835	6·68	6·95	484 } 2,840	555 } 2,355	577 } 2,167
		223	2·470	1·827	90·3	0·0324		0·3188			9·32	8·92		2,080	1,990
												Average flow	3,855	2,845	2,775
11	3,090	355	5·80	3·227	61·2	0·0405	0·2012	0·3434	0·060	16·07	17·73	13·67	5,700	6,290	4,850
		58	1·00	1·000	58·0	0·0405	0·2012	0·3434	0·060	4·98	5·49	5·675	289 } 5,989	318 } 6,608	329 } 5,179
		413	3·464	2·289	119·2	0·0405		0·3434			12·58	10·57		5,190	4,360
		206	4·10	2·560	50·2	0·0300	0·1732	0·3107	0·060	10·98	12·72	11·50	2,260	2,620	2,366
		240	2·40	1·793	100·0	0·0300	0·1732	0·3107	0·060	7·69	8·90	8·80	1,845 } 4,105	2,138 } 4,758	2,111 } 4,477
		446	2·970	2·066	150·2	0·0300		0·3107			10·28	9·785		4,580	4,365
		263	3·76	2·418	69·9	0·0430	0·2074	0·3503	0·060	12·40	13·54	11·20	3,260	3,560	2,945
		70	1·00	1·000	70·0	0·0318	0·1783	0·3168	0·060	4·41	5·06	5·675	309 } 3,569	354 } 3,914	397 } 3,342
		333	2·380	1·783	139·9	0·0407		0·3439			9·795	8·750		3,260	2,916
												Average flow	4,554	4,343	3,880
12	5,630	610	8·50	4·165		0·0352	0·1876	0·3277	0·070	16·60	21·83	16·56	10,120	13,320	10,100
		651	8·10	4·035		0·0257	0·1603	0·2951	0·070	13·73	19·04	16·16	8,930	12,400	10,520
		488	7·90	3·968		0·0356	0·1887	0·3290	0·070	15·88	20·86	15·95	7,740	10,180	7,780
												Average flow	8,930	11,967	9,467

† Velocity probably hypercritical.

where W denoted the surface width. That was slightly greater than what might be termed the Boussinesq critical velocity, given as :

$$V_B = \sqrt{gR} = \sqrt{\frac{gA}{P}}.$$

The relation was $V_C = \sqrt{\frac{P}{W}} \cdot V_B$, in which the multiplier $\sqrt{\frac{P}{W}}$ was in most cases little greater than unity. That being so, and since he did not know the values of W, in Tables 3 and 4 he compared values of Q_M, calculated from Manning's formula, with corresponding values of Q_L and Q_B calculated respectively from Lacey's formula, $V_L = 16(R^{\frac{3}{4}}S^{\frac{1}{2}})$, and the Boussinesq critical velocity, $V_B = \sqrt{gR}$. Where the Authors had shown the cross-section subdivided into a deep and shallow portion, Tables 3 and 4 showed values of Q_L and Q_B calculated for each portion separately ; but, in the final comparison for each gauging site, values Q_L and Q_B, taken in averaging, were those for the whole cross-section, assuming that the R for the whole section was given by dividing the whole area by the sum of the two sub-perimeters.

Comparison of Q_M with Q_L and Q_B was of course valid only if the section considered was lined with active detritus. In certain exceptional cases, it appeared that that condition was not fulfilled, but that the bed consisted of a slab of exposed bed-rock long enough and smooth enough for hypercritical flow to persist and leave its corresponding wrack-lines. A typical case of such action appeared to be gauging site No. 7 at Lyn Rock, for which Mr Chacinski had taken unusually low values for n.

Finally, it was of interest to note that for Lacey's fundamental formula, $V = 1{\cdot}1512\sqrt{fR}$, to give the same value for V as $V_B = \sqrt{gR}$ required f to have a limiting value of 24·3—a value very close to that which Mr Crump had been given to understand was contemplated for adoption in designing certain new channels in the vicinity of Lynmouth.

Dr John Glasspoole said that the Meteorological Office had been in a better position to define the rainfall of 15th August, 1952, because it had just started an investigation into the areas of the greatest average rainfall from the Quantocks to Exmoor. The initiative had come from Mr C. H. Archer, whom the Authors had mentioned.

Following a visit he had made to the proposed sites in May 1951, the Meteorological Office had lent nine rain gauges, five on sites from Dunkery westwards and four on sites on the hills to the east. As a result of Mr Archer's keenness, they would certainly also get more detail into the map of average annual rainfall which was mentioned on p. 531. He thought they would find that there was a 65-inch line on Dunkery and probably a 70-inch line over the highest ground around the Chains and near Kinsford Gate.

In spite of their special interest in the area at the time, in spite of other observers in the area, and in spite of the activities of the Somerset River

Board, it was clear that they could have used two or three times the number of records in drawing the map in *Fig. 9*. One reason for thinking that the distribution on that map was quite a good general picture was that little change had been necessary in positioning the lines following the visit by Mr Bleasdale to the area. He felt that if there had been more records, whilst some areas might have been wetter than those shown in *Fig. 9*, other areas undoubtedly might have been drier.

During such abnormal rain, it was clear that a close network of rainfall stations was desirable, but those storms were rare at any one place. The average observer—bearing in mind that there were more observers in drier parts of Britain than in wetter parts—could expect to record 2 inches in 1 day once in 9 years. He could expect to record 3 inches in a day once in 60 years. In point of fact, on 18th August, 1952, the observer at Hastings had recorded 2·47 inches, which was the largest amount in 1 day which had been recorded there for 78 years. So there was little incentive, therefore, afforded to an observer by his chance of measuring some sensationally large daily amount.

Moreover, a vast increase in the number of rainfall observers, which was already 5,000, could hardly be justified to deal with such infrequent heavy rains. One needed only to imagine planning adequately to record the next Lynmouth flood, which might occur in 1953 or in, say, 200 years' time !

He wished to ask the Authors what they thought should be done to prepare for recording further storms. A reasonable solution to the problem, to his mind, seemed to be to fill up the large gaps in the distribution of the rainfall stations in the more mountainous districts, with the aid of the river boards ; to maintain a close network of stations in some localities, such as around London, where the incidence and the distribution of heavy rains and the synoptic situations could be studied in some detail, in order to define the general patterns which could be applied to other storms ; to encourage observers to make a special effort to get readings of more unusual rains ; to encourage authorities to keep recording rain gauges in good order against the occurrence of abnormal but infrequent storms ; and to seize every opportunity of comparing rainfall and run-off during such abnormal storms, including the use of models, in the hope of making generalizations which could be applied and used with other storms in other areas.

On p. 539 it was argued that if the Bruton storm of 28th June, 1917, had occurred over the Lynmouth area, the peak flow would have been greater than that of August 1952 ; but he felt that that argument was overstated. In any case, the storm of August 1952 had given a larger area with more than 9 inches, a larger area with more than 8 inches, and a larger area with more than 7 inches than the corresponding storms of Bruton or Cannington, taking the whole storm and not that confined to the Lyn catchment. The areas covered by the earlier storms had been

given in a Paper [19] which he had presented to the Institution in 1929. Moreover, in the Bruton storm practically all the rain had been spread over 16 hours, with an estimated peak of 1 inch per hour in the central area. In the Lynmouth storm, about three-quarters of the rain had been confined to 7 hours, so that presumably the Lynmouth storm had been also more intense. He asked the Authors whether their statement on p. 539 merely meant that if the heaviest rain of the Bruton storm had been so distributed that it was centred over the Lyn catchment, greater floods would have occurred. Surely the chance of the heaviest rain being centred precisely over the catchment area was very remote. It was known that in the storm of 15th August, 1952, the heaviest rain had not been centred over the Lyn catchment area, as shown in *Fig. 9*, and that was the common pattern in the rainfall distribution.

He hoped that the Authors would set out from their experience what should be done now in order to record future storms more precisely.

Mr G. W. Green said that he wished to enlarge briefly upon three points raised by Mr Wolf in quoting from the script of his report which would be published in a forthcoming Bulletin of the Geological Survey.

First, referring to p. 541, boulder beds in the alluvial deposits of all the rivers showed that intense flooding had occurred on many previous occasions. The most spectacular evidence was a large boulder dump partly blocking the East Lyn gorge about half a mile upstream from Lynmouth, that had been partly opened up by the floods, exposing boulders which were even larger than those recently carried by the East Lyn. The volume of material remaining (about 15,000 cubic yards) was overgrown by vegetation including oak trees probably 60 to 80 years old ; it was possible that that dump had been caused by the flood of 1769 mentioned on p. 541.

Secondly, referring to p. 533, the reasons that had made him suggest the storm centre to be at the headwaters of the Hoaroak Water and West Lyn (main river) had been the difficulty in explaining the very large flood-level cross-sections and boulder spreads in the West Lyn and Hoaroak and Farley Water valleys, which were considerably greater than in the Shallowford valley. The pattern of most intense downcutting at the head-waters of the various rivers corresponded approximately to the 8-inch isohyet shown by Bleasdale and Douglas. Equally intense downcutting had occurred, however, at favourable places over a wide area beyond the head-waters and showed the importance of local factors apart from rainfall —in particular the gradient and the nature and thickness of the superficial deposits—in determining those sites.

His last point related to p. 530. Tremendous boulder movement had occurred during the floods, and the rivers had completely choked their channels in numerous places, particularly at bends, and cut much straighter channels by-passing those places. In Lynmouth itself, the filling-up of

[19] "The Areas Covered by Intense and Widespread Falls of Rain." Min. Proc Instn Civ. Engrs, vol. 229 (1929–30, Pt I), p. 137.

the space (about 12 feet high) under the bridge spanning the West Lyn by boulders had been a prelude to the diversion of the river down the High Street.

His survey of the North Exmoor flood area had shown that the boulder movement had been related to the geology as well as the various hydraulic factors. The volume of boulders shifted by the rivers had differed sharply according to the nature of the superficial deposits and those, in turn, with the solid geology underneath. The different types of solid rock outcropped in parallel bands across Exmoor.

The Ilfracombe Beds and Morte Slates were relatively soft and readily eroded into small chips and slabs, whilst the grits and gritty slates of the Lynton Beds and Hangman Grits were tough and blocky and survived as very durable boulders. Thus superficial deposits overlying, and derived from, the shales were usually fine-grained and contained relatively few slabby boulders, whilst those derived from the grits were very bouldery. The bedrock was very resistant to erosion and only the shales and slates were affected in a few places, but the drift deposits were very readily attacked. Therefore, in stretches of river comparable in every other way the volume of boulders moved had been immensely larger in those flowing over grits than in those flowing over shales and slates.

Mr V. K. Collinge said that he wished to comment on the remarks made by Mr Wolf on the times of concentration of the East and the West Lyn. By consideration of the velocities of flow in the rivers, Mr Wolf had suggested the time of concentration of the rivers to be 3·5 hours for the East Lyn and 2·0 hours for the West Lyn. By using the Bransby-Williams formula,

$$T_c = \frac{l}{d} \times 5\sqrt{\frac{A^2}{f}}$$

where l denoted greatest distance from watershed to the outfall, in miles ;
A ,, catchment area, in square miles ;
d ,, diameter, in miles, of circle with area A ; and
f ,, average fall, in feet, per 100 feet,

figures of 7·1 and 3·1 hours had been obtained. Substituting those figures back in the formula, it appeared that the Authors had used for l the distance from the farthest point on the watershed to the outfall, including all the river meanders.

It was to be questioned whether that was the meaning intended by Bransby-Williams. Both in his original Paper [20] and in his book entitled " Storage Reservoirs," [21] Bransby-Williams had defined l as " the greatest distance from the watershed to the outfall." That might equally well be

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[20] " Flood Discharge and the Dimensions of Spillways in India." *Engineer*, vol. 134, p. 321 (29 Sept. 1922).

[21] Chapman & Hall, 1937.

taken as the direct distance from the farthest point on the watershed or as the distance along the river course. It was to be noted that the value of l also affected the value of f in the formula.

Using l = the direct distance = l^*

$$f = \frac{\text{maximum height of watershed above outfall}}{l \times 5{,}280} \times 100$$

$$= f^*$$

a time of 4·5 hours was obtained for the East Lyn and 2·1 hours for the West Lyn.

A further point to bear in mind was that in the Institution Interim Report on Floods it was stated quite definitely that, for upland areas, the Bransby-Williams formula should be used with $f = 3{\cdot}3$ and $\frac{l}{d} = 1{\cdot}2$. Using those figures, one obtained values of 3·7 hours for the East Lyn and 2·3 hours for the West Lyn.

Finally, estimates had been made using the Richards formula. Again, l was not defined exactly, but Richards had suggested using the distance from the edge of the catchment excluding minor river meanders. Using appropriate values for the other coefficients, the following figures were obtained: East Lyn, 3·5 hours; West Lyn river, 1·3 hour; West Lyn tributary, 1·7 hour (that was, 1·7 hour for the West Lyn catchment).

TABLE 5

	T_c (actual)	T_c (Bransby-Williams)	T_c (Bransby-Williams with l^* and f^*)	T_c (I.C.E.)	T_c (Richards)
East Lyn .	3·5	7·1	4·5	3·7	3·5
West Lyn .	2·0	3·1	2·1	2·3	1·7

From the figures in Table 5 the following conclusions might be drawn. First, the Bransby-Williams formula had to be treated with caution until some clarification of the meaning of the length of the catchment was forthcoming. Secondly, for the catchment in question the Bransby-Williams formula, used as recommended in the Institution Report, and the Richards formula, when used with care, both gave good estimates of the time of concentration.

Mr Julius Kennard referred first to a statement made by Mr Dobbie on p. 529, which read: "The possibilities of afforestation were studied but it was concluded that under the worst conditions it would not make a material contribution to flood alleviation." He wondered if the Authors could elaborate upon that, because the impression obtained from reading

it was that the Authors believed the effect of afforestation to have no bearing on the intensity of flood discharge from the catchment. He was hoping that Mr Dobbie would say that that had not been his intention, because those who had to concern themselves with the operation of the Reservoirs (Safety Provisions) Act often found themselves in difficulty with regard to the intensity of flood discharge that might come from a catchment draining to a comparatively small reservoir.

It was well known that in Great Britain there were numerous small reservoirs used perhaps as fishponds, boating ponds, and so forth, and generally owned by impoverished country gentlemen who, under the Reservoirs Act, found themselves liable to be called upon to carry out rather extensive alterations and repairs to deal with inadequate overflow works, and one hesitated in those circumstances to adopt the Institution Flood Committee's Report giving the intensity of flood, without some qualification. Generally speaking, one tried to examine the catchment to ascertain what percentage of afforestation there was upon it. He wondered if the Authors, with their experience, could give any guidance in that connexion. Could they give a figure for the percentage reduction in the intensity of flood discharge in an area of fairly dense afforestation? Mr Kennard felt sure that, in the Paper, Mr Dobbie had only meant that it was because of the exceptional physical conditions of that catchment and the presence of boulders that afforestation had been studied but disregarded.

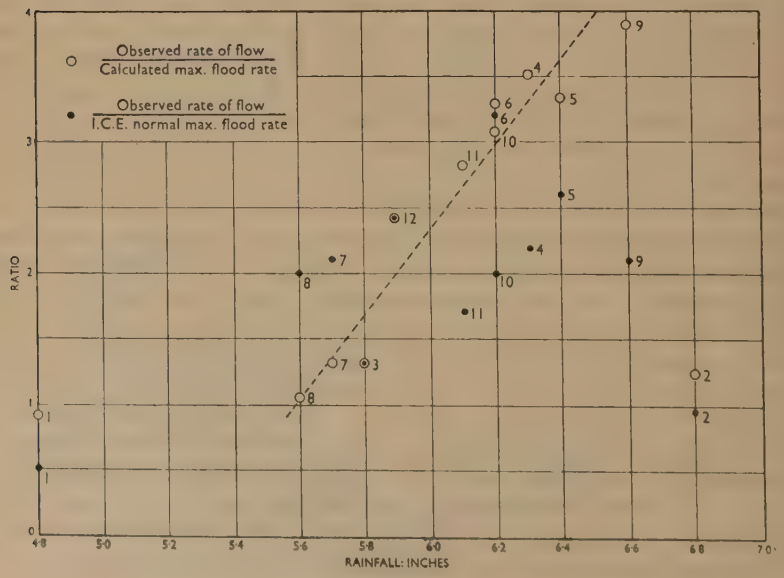
The second point related to Fig. 8, Plate 2, which gave the catchment on a scale of 1 inch to 1 mile. He suggested that it would be of special interest to hydrologists to have the rainfall lines on *Fig. 9* transferred on to Fig. 8, Plate 2. Looking at those lines, he had tried to seek confirmation of something about which he had had a bee in his bonnet for many years, and which went back to the time when the Institution Committee had been considering its Interim Report. At that time he had made a suggestion to the Committee that the intensity of flood discharge depended to some extent (there were, obviously, numerous other factors) on what he had called the average height of the catchment. He had tried to define that term as the average height of the ground above the point at which the flood was measured, and to do that, of course, one needed the contours of the catchment, which unfortunately were not all given on Fig. 8, Plate 2. He thought that he had found some confirmation of that theory at Lynmouth, because it could be shown from the Figure that, in the case of the gauging stations 4, 5, and 10, where there were long narrow valleys the heads of which were above 1,500 O.D., there were flood intensities of 2, 2·4, and 2·6 times the Committee's recommendation for normal floods. Moreover, the rainfall on those three valleys seemed to be about the highest over the whole of the catchment. On the other hand, on the east of the catchment, it was found that with a broad, flatter (one could not call it flat) valley, the intensity was only half of the Institution's recommendation.

From the Ordnance Survey maps of the several catchment areas, and the computation of the rainfall which Mr Bleasdale had prepared, in each of the areas above gauging stations Nos. 1 to 12, Mr Kennard had proceeded on the basis of the formula $Q = \frac{K}{\sqrt[3]{M}}$, without accepting that the value of K should be 750, as put forward by W. J. E. Binnie and H. Lapworth in 1928. He (Mr Kennard) had always considered that it was a mistake to ignore the effect of shape and slope, and to regard the intensity of flood as being governed solely by the area of the catchment. The value of K should depend on the shape and slope of the catchment, and by ascertaining the "average height" above the point of concentration, it was believed that those characteristics were taken into account, and more reliable values of K would be arrived at.

A study of a number of catchments in different parts of Great Britain with which Mr Kennard was familiar, and for which he had records of floods, had enabled him to develop a basis for arriving at the value of K for areas of different shapes and slopes, and therefore different average heights.

The areas above gauging station No. 12 at Lynmouth had been analysed accordingly, and Table 6 indicated the "average height" and the appropriate value of K as deduced from a straight-line curve having the value of 1,025 for K, corresponding to an average height of 750 feet, and passing through the origin. The calculated flood was then compared with the

Fig. 15



flood observed by the Authors, and the ratio of the latter to the former was shown in the last column. It should not be overlooked that the comparison was between a catastrophic flood and a normal flood.

With the object of comparing the intensity of flood ratio with the rainfall, on each of the areas, Mr Kennard had prepared *Fig. 15*, and it would be seen that there was a tendency for the points to fall along a curve. If the " average height " factor was ignored, the ratio of the observed rate of flow to the I.C.E. normal maximum flood rate, as set out in the Authors' Table 1, could also be plotted against the rainfall (*Fig. 15*), but in that case the comparison did not appear to be so readily related.

TABLE 6

No. of gauging stations	Rainfall : inches	Average height : feet	Value of K adopted in formula $Q = \frac{K}{\sqrt[3]{M}}$	Calculated flood : cusecs per 1,000 acres	Observed flood : cusecs per 1,000 acres	Ratio of observed rate of flow to calculated maximum flood rate
8	5·6	1,050	1,450	732	770	1·05
7	5·7	895	1,230	623	820	1·31
6	6·2	648	890	745	2,410	3·24
12	5·9	625	860	654	1,580	2·42
3	5·8	585	800	463	610	1·32
5	6·4	540	740	792	2,640	3·34
4	6·3	470	645	690	2,420	3·51
10	6·2	450	620	663	2,040	3·08
2	6·8	437	600	438	550	1·25
11	6·1	410	565	523	1,470	2·81
1	4·8	365	500	399	360	0·90
9	6·6	365	500	583	2,270	3·90

Mr Gerald Lacey observed that, with regard to reservoirs and dams, there was an Act which required that they should be inspected occasionally, and there was a panel of engineers whose business it was to determine whether those dams were constructed with reasonable regard to human safety. It had been disasters and casualties which had caused that Act to be passed, and engineers now had to take action because they had the panel.

It appeared to him that, with torrential rivers of the character of the Lyn, with a somewhat unpredictable flood, no major highway bridge should be constructed without prior reference to some competent engineering body for determination whether in fact the waterway was sufficient or not.

In addition to the formula referred to by Mr Crump, he had produced a very simple equation concerning the waterway and the maximum discharge anticipated. He had not been able to compare that with the

waterway of the bridges in question, but of one thing there was no doubt whatever, namely, that the waterway of those bridges at Lynmouth had been quite inadequate for the maximum discharge they had to carry.

He did not know what computations had been made with regard to the waterway of the bridges. It had to be remembered, also, that those intense floods were very rare and were in the nature of cataclysms and catastrophes. As a general rule, the highway bridge engineer in Great Britain confined himself to what he could see with his own eyes. Mr Lacey had no doubt that the bridge had been designed for, and had carried happily for many years, the normal discharges. Nevertheless, where human life was at stake, it was necessary to legislate for the worst that could happen, or something not far from it.

He suggested seriously that engineers, particularly those concerned with hydraulics, should give attention to how a disaster of that sort could be averted either at Lynmouth or elsewhere in the future.

As for trying to reduce the flood intensity at a place like that by afforestation, admirable though that might be, it might indeed be possible to effect a reduction of, say, 10 per cent ; but the next time, luck might be out, another record might be broken, and the discharge might be 10 per cent higher than it had been before !

It had been suggested by Mr Crump that his (Mr Lacey's) regime equation, $n = 0{\cdot}093\, S^{\frac{1}{4}}$, might have been employed as a guide to the value of Manning's n employed on the Lyn. Unfortunately, although the Authors' sites had been selected for straightness in plan and uniformity in cross-section, it was clear that on the Lyn the value of n depended not only on the general roughness and size of the bed material but also on that exceedingly elusive characteristic usually termed " channel condition." That gave full scope for the judicious application of what was effectively a " condition factor " to what might be termed a *normal* value of Manning's coefficient n.

From a Table with which the Authors had kindly supplied him, values of n and corresponding slopes had been abstracted as given in Table 7. All the slopes were of the same order.

TABLE 7

Site No.	S	n	River	Gauging station No.
1	0·0170	0·029	East Lyn	7
2	0·0175	0·045	Oare Water	1
3	0·0186	0·045	Farley Water	7
4	0·0200	0·0435	East Lyn	7
5	0·0210	0·057	East Lyn	8
Average	0·0188	0·0438		

At the first site in the Table, the bed appeared to be inordinately smooth, and at the last to have been in very bad " condition."

The Authors had referred to surges, and the hydraulic data required re-examination in respect of those sites of which the assessed velocities were greater than $(gD)^{\frac{1}{2}}$.

The Ryves equation for maximum discharges, $Q = CM^{\frac{3}{4}}$ (in which Q denoted the discharge in cusecs, C a coefficient, and M the catchment area in square miles), appeared to give fair values for a catastrophic flood with a coefficient of about 2,000.[22]

Mr T. M. Chacinski said that he wished to enlarge upon the method which had been adopted for assessing n and c, with which part of Mr Dobbie's work he had been actively though not directly connected. As Mr Crump had said, it had not been a very easy task, and he agreed with him that some of the formulae which did not take roughness into account could be much better used than the other method, and he knew of some Continental formulae which, as a later check, might be said to give quite good results. The first was Matakiewicz's formula :

$$v = 35{\cdot}4\ m^{0{\cdot}7} i^{(0{\cdot}493\ -\ 10i)} \text{ metres per second}$$

where v denoted mean velocity,

m denoted hydraulic radius in metres, and

i denoted the slope.

That formula was well quoted in German and Austrian sources and it had been developed from a very large number of measurements on rivers and torrents. It was unique in that influence of slope vanished at high slopes. It could be reduced to the form $v = \Psi(m)\Psi_1(i)$ metres per second and easily solved by special Tables or nomograms. The second similar formula, also well known on the Continent, was Winkel's formula :

$$v = m^{\frac{5}{7}} i^{\frac{4}{7}} \left(185 - 210 i^{\frac{0{\cdot}5}{7}}\right) \text{ metres per second}$$

Mr Chacinski had often applied the first of the above formulae during his work before the war ; in the present case, however, because it had been felt that those formulae might not be quite suitable, the following method had been adopted.

As a matter of fact, there was a remarkable lack of good evidence as to what was the value of c in a channel of the roughness and steepness of the Lynmouth torrents. Such channels behaved differently from channels with a mild slope, probably because they formed a series of constrictions which at critical stages made them independent of the slope. In those circumstances any formula of Chezy or Manning form was no more than a sort of shorthand, for the circumstances were not quite expressible in such terms. However, those formulae could be a useful way of presentation.

[22] W. J. E. Binnie and Herbert Lapworth, " Floods, with special reference to Waste-Weir Capacity." Engng Conf., 1928. Instn Civ. Engrs. *See* p. 147.

The procedure had been to adopt two roughness formulae for c, one the formula of Professor White and the other a variation of Prandtl's formula which was quoted by Professor White and Dr Nemenyi. The first formula was :

$$c = 32 \log_{10}\left(13 \times \frac{d}{k}\right) \text{ feet}^2 \text{ per second}$$

where d denoted mean depth in feet, and
k denoted size of bed material in feet.

The second formula was :

$$c = 32g \log_{10}\left(0{\cdot}4 \frac{m}{k_1}\right)$$

where m denoted hydraulic radius and $k_1 = \frac{1}{10}K$, where K denoted size of boulders.

That had been one basis ; another had been to look at any reach of the river for a well-developed, measurable, and accessible critical section which could be regarded as a proper constriction, and to check the flow by a critical discharge formula :

$$\frac{A^3}{B} = \frac{\alpha Q^2}{g}$$

where A denoted area of cross-section, and
B denoted width at water level.

The coefficient α was a correction for curvilinear distribution of velocity and adoption of a value was a difficult point. Mr Chacinski supposed, however, that those values of α should not be much less than 1·2 and not much more than 1·5, so that error would not be very great.

Unfortunately, very often it was not possible to find a cross-section which could be regarded as a constriction, so that then only the first part would be applied. As an additional check, however, one of the formulae quoted by Professor Allen was applied, based on the observation of rows of boulders, or rather concrete blocks which would be moved at a certain mean velocity. That would give some idea of the range of the velocities which might occur in a given reach. The formula was :

$$\bar{v} = 1{\cdot}2 \frac{L}{l}\left(10 - \frac{5{,}300}{\left(32 - \frac{h^2}{l}\right)}\right)\sqrt{\frac{\zeta' - \zeta}{\zeta}} \text{ feet per second}$$

where $\bar{v}$ denoted mean velocity at the section which would just move the uniformly scattered concrete blocks
L and l denoted dimensions of blocks in feet
h denoted depth of water above blocks, in feet
ζ' and ζ denoted specific gravities of blocks and of water.

ζ' was assumed to be 2·5 for hard stone grit. By comparing the results of several checks by the adopted method, a value of n could be devised.

Mr Chacinski mentioned that Professor White's formula had been applied in the reaches where the boulders were small and closely packed, and the variation of Prandtl's formula had been applied in reaches where the boulders were very large and scattered. Those two formulae had to be used in their appropriate places.

The values of n which had been obtained were 0·04 to 0·08. He had a feeling that those values were slightly under-estimated. He supposed that, by applying a formulae such as that of Mr Lacey, or one of the Continental formulae quoted above, one might get results which would be closer to reality, because in fact his supposition was that the flow had been slightly over-estimated. He did not think that the error was greater than 10 per cent in general, but at some cross-sections that might be so. However, the final error had been very nicely checked by the model which the Authors had mentioned. At the spot, the flow at the zigzag constriction had been estimated at 8,500 cusecs, and the Authors had measured 7,800. That was the order of error which occurred in all those calculations through assessing n values.

*** **Mr W. N. McClean** asked how it could be shown that there was any degree of accuracy in the estimate of flow in the selected reach. Was the value anything more than a momentary peak discharge? Such an estimate was based on an after-survey which assumed probable water levels in that reach. Actually, it was a procedure which avoided the work of survey and record.

It was really saying that flow could be deduced from the formula $v = c\sqrt{mi}$, with momentary values of c, m, and i. There was no attempt to discover what the flow had been throughout the rise and fall of the flood.

It was, in fact, available storage which governed the rate of discharge of the inflowing rainfall, and continuous record alone provided values of storage. Practically speaking, on that 40 square miles, there was no basic water survey or record.

Mr David Lloyd * referring to the effect of afforestation of an area on the run-off, recalled a Paper [23] he had read at the British Association meeting in 1950. In that Paper he had reached the following conclusions by statistical inference from the records of the Vyrnwy (Montgomery) catchment of 23,290 acres, where the acreage afforested had been increased from 3 per cent to 20 per cent of the catchment: first, that over a long period of years, afforestation had no discernible effect on the quantity of long-period run-off; secondly, that there was no evidence to indicate

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*** This and the following contributions were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

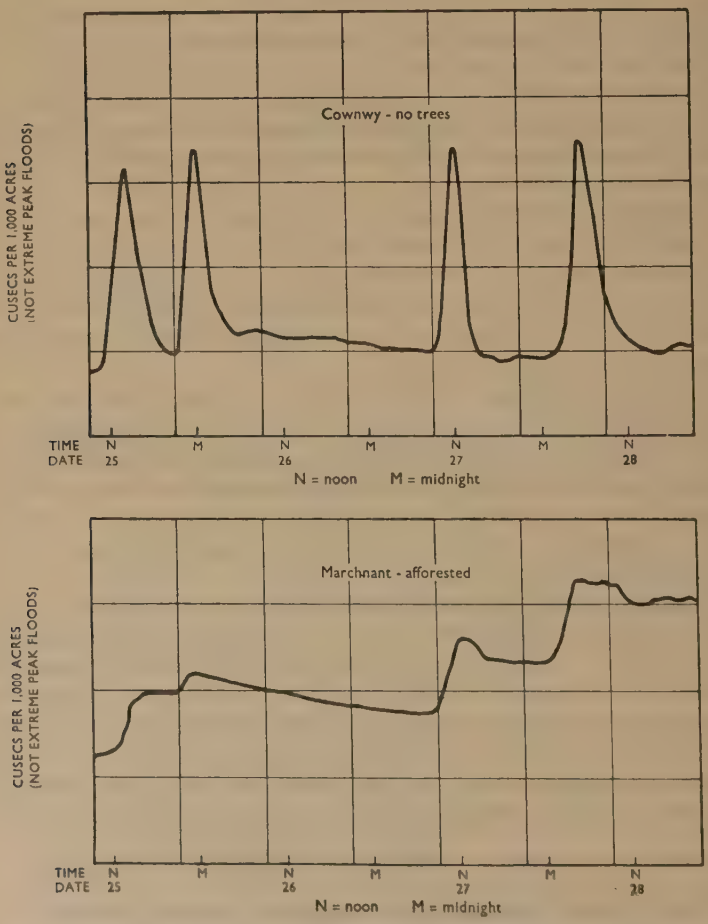
* This contribution was made with the permission of the Chief Engineer of the Liverpool Corporation Water Works.

[23] Brit. Assocn Mtg, Birmingham, 1950.

that the *extreme* peak floods had been modified. It was now possible to show the effect of afforestation on the lower peak floods.

As part of the whole Vyrnwy drainage area there were two subsidiary areas, the Cownwy area (3,200 acres) and the Marchnant area (1,828 acres); on the Cownwy very few trees had been planted (3 per cent of area covered), whereas the Marchnant was largely afforested (60 per cent covered); at both areas there was a continuous hydrograph made at the diversion point. Those areas were geologically similar and sufficiently similar in other respects for a comparative examination of their hydrographs to show the effect of afforestation modifying flooding when a storm

Figs 16


Cownwy - no trees
CUSECS PER 1,000 ACRES (NOT EXTREME PEAK FLOODS)
TIME N M N M N M N
DATE 25 26 27 28
N = noon M = midnight
Marchnant - afforested
CUSECS PER 1,000 ACRES (NOT EXTREME PEAK FLOODS)
TIME N M N M N M N
DATE 25 26 27 28
N = noon M = midnight

Comparative Hydrographs for Cownwy (no Trees) and Marchnant (Afforested) for Period during October 1952

passed over. When an isolated storm passed over, the hydrograph at the treeless area reached a peak rate of discharge in accord with the intensity of the rainstorm ; when the rain ceased, the rate of discharge fell rapidly and the period of the falling curve was not much longer than the period of concentration. From the afforested area the peak rate of discharge appeared to be considerably modified—ranging up to half the treeless-area peak—but the falling curve fell very slowly. That suggested that the plantations created a pondage impounding part of the surface flow followed by its slow release.

That natural storage had a marked effect on the hydrographs produced by consecutive storms, depending on the time interval between storms ; for in the case of the Cownwy area a series of short sharp floods occurred whereas at Marchnant the peak was damped but the flood period lengthened and a following flood imposed on the tail discharge. Eventually, in a series, the storage was full and the peak discharge was similar to that from the treeless Cownwy area. The natural storage of plantations was shown clearly in the simplified graph of the October 1952 period (*Figs 16*). A rain spell had commenced on the 21st October giving consecutive storms estimated to have similar rainstorm intensities at each sub-area. The hydrographs clearly showed at the afforested area the damped-down peaks, the capacity of the natural storage being utilized, and then the peak discharge rate being similar to that from the treeless Cownwy area.

Such repetition of storms was rare ; it resulted, at 4 a.m. on the 28th October, 1952, in a peak discharge from the whole area (23,290 acres) of 224 cusecs per 1,000 acres, a value less than but quite consistent with the extreme peaks experienced at that area and in accord with the average intensity of storm rainfall from the wet catchment.

The conclusion was reached that afforestation by its storage effect would dampen down the peak discharge of an isolated storm but not the later peaks of a series of storms, and would prolong the period of flood discharge.

Mr B. D. Richards observed that the analysis of the Lynmouth flood of August 1952 presented great difficulties owing to the lack of sufficient recorded data, but the evidence that the Authors had collected showed beyond any question that it had been a " catastrophic " flood of a high degree.

They had endeavoured to estimate the peak flood in two ways : first, from the cross-sections of the river and the flood levels as determined from the deposits of wrack ; and secondly, from the rainfall.

The first estimate was, as the Authors had shown, an uncertain one in a river such as the Lyn, owing both to the difficulty in assessing the coefficient n and also to the surges produced by temporary blockage of the channel which might lead to an erroneous determination of the gradient.

In that particular case, a determination of the peak flood from the rainfall might be a more promising line of approach.

The Institution of Civil Engineers Floods Committee's curve for normal maximum floods showed peak intensity plotted against catchment area for typical upland catchments and was based on a rainfall coefficient $R = 4$. For the 25,000-acre catchment of Lyn, that gave a peak flood of 8,250 cusecs. A catastrophic flood for the same area of catchment, based on $R = 8$, would give a shorter period of concentration and a peak of 23,350 cusecs.

The curve did not provide for variation of the characteristics of an individual catchment, such as shape, slope, coefficient of run-off, etc. and probably gave too high values for many upland catchments, but it gave too low values for others such as in the West Highlands of Scotland.

Mr Richards had endeavoured to estimate the maximum catastrophic flood of which the Lyn was capable and the peak value of the actual flood of August, using the methods set out in his book.[24]

For England, the coefficients of rainfall used were $R = 4$ for normal maximum floods and $R = 8$ for catastrophic floods. For the West Highlands of Scotland, $R = 6$ and 12 were advocated. For the Lyn catchment, coefficients of $R = 5$ and 10 would appear appropriate.

The characteristics of the catchment had been taken as :

area = 25,000 acres ; $f(a) = 0{\cdot}73$; $L = 8{\cdot}8$ miles ; $S = 0{\cdot}032$; $K = 0{\cdot}80$

For $R = 5$	$KR = 4$	$C = 0{\cdot}010$
For $R = 10$	$KR = 8$	$C = 0{\cdot}008$

Using those coefficients, the following figures were obtained.

	Q	Period of concentration
Normal maximum flood =	17,000 cusecs	3·29 hours
Catastrophic flood =	45,600 „	2·20 „

The peak intensity of the flood depended on the time distribution of the rainfall. The highest and lowest peaks were obtained with rainfall diagrams giving factors $n_5 = 0{\cdot}720$ and $n_5 = 1{\cdot}777$ respectively.[25] The peak intensity and period of concentration would then vary as shown in Table 8.

TABLE 8

	n_5	Peak intensity : cusecs	Period of concentration : hours
Normal maximum flood	1	17,000	3·29
	1·777	13,900	4·26
	0·720	19,000	2·84
Catastrophic flood	1	45,000	2·20
	1·777	38,000	2·83
	0·720	50,150	1·91

[24] " Flood Estimation and Control." Chapman & Hall, 1950 (2nd edn).

[25] *Ibid.* See Table 20, p. 84. Maximum intensity, *Fig. 9*, $n_5 = 0{\cdot}720$. Minimum intensity, *Fig. 12*, $n_5 = 1{\cdot}777$.

The data were inadequate for an exact determination of the actual flood of August.

The highest spot rainfall was taken as 6·4 inches in 4 hours, based on the Meteorological Office interpretation of the Longstone Barrow gauge reading of 5·9 inches in 4 hours (*see* p. 533), and an indication that the rainfall was somewhat higher elsewhere.

That was equivalent to a value of $R = 8$, which would give a peak flood of 23,350 cusecs with a period of concentration of 2·42 hours. That figure might be compared with the Authors' estimate of 22,000 cusecs (see p. 539).

It was therefore suggested that the normal maximum flood for the Lyn was of the order of 14,000 to 19,000 cusecs, average 17,000, according to the time distribution of the rainfall; and that the maximum catastrophic flood was similarly of the order of 38,000 to 50,000 cusecs, average 45,000 cusecs.

The August flood therefore appeared to have been of the order of 40 per cent above the normal maximum flood and about one-half of the maximum catastrophic flood of which the catchment was capable. Mr Richards's assessment of the actual peak flood of August 1952 was in close agreement with the figure found by the Authors, but that of the maximum catastrophic flood of which the catchment was capable, was considerably higher than theirs.

Mr F. T. Walton observed that civil engineers accepted as part of their mission in life that they would be called upon at short notice to stem and repair the havoc wrought by the elements. But there was another enemy, apart from the elements, which was largely ignored; that was, public apathy and the short span of human memory of the details of disasters such as the Lynmouth flood. He believed that the Authors had had that point in mind when stating, on p. 529, "It is possible that these recommendations will not be accepted, on the grounds that Lynmouth is not worth the expenditure and that the inhabitants are opposed to a large safe channel."

Did the engineering profession feel that enough was being done to make the powers-that-be aware of that danger, which prevented the execution of schemes to reduce the menace of future tempests?

Mr R. H. MacDonald observed that, for cases of cataclysmic floods, it was probably impossible to have established beforehand, in Great Britain, any method of measuring the actual peak flows and, of course, during the height of heavy storms, it was impossible to find anyone free to examine the question on the spot. The Authors were to be complimented on having been able to make estimates of what occurred at Lynmouth from site observations soon enough after the cataclysm had, for the most part, subsided.

The Authors had referred to Manning's formula. Had the Authors assumed that the surface slope of the river at the chosen site was parallel

to the bed slope. That was disclosed only after the flood had subsided and might or might not give a proper indication of conditions during violent active bed conditions. The surface slope could hardly be assumed from the general detritus line unless the river flow was uniform. The detritus line would show the height of the wave crest as it passed down, but not its actual surface slope.

Over short reaches, the flow would accelerate under gravity, and the bed slope and surface slope could not be parallel, and when the limit was reached, the high velocity would cause complete breakdown of the bed. Slowing up as the wetted section was increased would cause temporary deposition of bed load, forming a small temporary dam. A river running at those gradients, with those heavy charges of water was, for the above reason, bound to run in spasms. Manning's formula could hardly apply, therefore, but if there was any continuity in Manning's formula at all, it was thought that a value for n could be arrived at for low stages which should be applicable, in some degree, to high stages, as a first approximation. The real difficulty was, therefore, not the value of n but the value of s and, of course, the value of R, which was very hard to judge in an active bed of large boulders.

On p. 531, approximate hourly distribution of rainfall from 4.30 p.m. was given. From those figures it should be possible to derive a curve for the rise and fall of the Lynmouth flood if some approximate run-off factor for the area could be assessed. The problem was, however, perhaps purely academic, since it was the peak flood that mattered.

It was felt that the Authors had come to the correct conclusion with regard to afforestation. When the forest was felled the ground would be extremely vulnerable to high run-offs, and there was no guarantee that peak storms would not occur at that time.

The question of the improvement in run-off after the storm had passed was very interesting. It would be interesting to know whether that improvement was gradually dying down again with the re-establishment of vegetation, and whether there was any marked difference between summer and winter run-off factors for the Lyn catchment area.

Mr C. H. Dobbie, in reply, observed that the Paper had evoked a gratifyingly lengthy discussion, containing material on many aspects of flood flows which had much value in itself and would, no doubt, form a useful basis for further work on the general subject. The Authors' assessment of flow had been subjected to exacting scrutiny by many leading hydraulic engineers and could be said to have stood the test remarkably well. There was perhaps a general feeling that the figures were just a little on the high side. Some contributors were not satisfied with the methods of measurement, but those also seemed to meet with general approval, since no practical alternatives had been suggested. In the rebuilding of Lynmouth, the Author's recommendations for sizes of channels and bridges had been adopted.

When one turned to geomorphology it was a very different story. The Author's version was accepted only by geomorphologists and completely rejected or ignored by engineers and laymen alike. The Author's recommendations, based on European practice of bed stabilization in the channels and the construction of check dams upstream, found no favour with the local or national authorities concerned.

Mr Thompson had asked about the relative levels of land and sea. During and after the recent glacial periods those relative levels had oscillated considerably; among the factors were the withdrawal of sea-water into the ice cap and the depression of the land by the weight of ice above. The general result was an uplift, but the last sizable movement had been a depression of land or a rise of sea level, often called the post-glacial depression, which itself contained oscillations. Mr Thompson would find a fascinating account in " Dating the Past : An Introduction to Geochronology," by F. E. Zeuner.[26] River capture was dealt with in " Landscape," by C. A. Cotton.[27]

Dr Glasspoole was anxious to receive suggestions on how he could improve his wholly admirable system of rainfall measurement. It depended on 5,000 enthusiastic amateurs of whom Mr Archer was a very worthy representative. The Authors felt that it was a very happy arrangement not to be meddled with by outsiders.

With regard to the water slope and to Mr McClean's observations about surges, the sides of the valleys were cut quite deeply and that top water-line was clearly defined in soil and vegetation, so that the flow had been held there for some time. In those circumstances, at the selected site it seemed fair to say that the measurements were of the water slope, and not of surges caused by temporary dams bursting. The sites had been specially selected to avoid those types of error and in general the results showed by their consistency that success had been achieved.

Mr Kennard's request for elaboration on the possibilities of afforestation were largely answered in the contribution by Mr David Lloyd. Clearly, afforestation could have beneficial results in damping down minor floods, but it had little effect on maximum floods. Since there were many minor floods to one maximum it clearly had considerable value in many catchment areas. Observations in Switzerland and America had shown that forests could absorb 3 inches of rainfall more than moorland. At Lynmouth, the whole of the area had been saturated before the rainfall that caused the flood, so afforestation (which was possible) would not appreciably have affected the flood flow although it would have alleviated soil erosion. Mr R. H. MacDonald had pointed out difficulties that follow the felling of afforested areas.

Mr F. T. Walton was concerned with public apathy to disaster, but the Authors felt that they could not answer for the engineering profession.

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[26] Methuen, 1950.

[27] Cambridge University Press, 1948.

The unsuitable siting of buildings in valley flood plains was a universal problem. Mr Richard's contribution should appeal to Mr Walton's sense of the dramatic, since he postulated a maximum flood twice as great as that of August 1952.

Mr P. O. Wolf, in reply, said that generally speaking the discussion had brought out a number of very important matters with which the Hydraulics Division would be concerned in future, one of the chief ones being the human aspect which hydraulic engineers generally had to bear in mind, possibly even more than other engineers. The disaster which had led to the work described had shocked everybody, and he felt that one of the reasons why the Authors had had so much help during their work, and why they had had such a kind reception, had been that the human sympathy of engineers and laymen alike had been involved to such an extent.

Another point that had come out in the discussion was that the accuracy claimed for the computations in that field was rather low, chiefly because he thought that hydraulic engineers were apt to be honest! They knew that the accuracy of their data was perhaps ± 20 per cent, and they did not like to claim a higher degree of accuracy for their results.

In reply to the Chairman's query on the effect of air entrainment in the river, and on its representation in the model, he agreed that aeration was one of the most difficult matters to reproduce faithfully in a reduced-scale model. In that respect, it had in fact not been possible to obtain exact similarity between the model and the natural gauging site; but in order to test whether the stage-discharge curve of the model (which was a calibrating device) would change through aeration, a surface-tension reducer had been added to the model-flood flow. This had resulted in some air entrainment, although not in proportion to the full-scale conditions, and had raised the surface level in the stream, but had had no effect on the readings of water level in the stilling well. It should be remembered that flow in the approaches to the gauging site was conducive to releasing some of the air entrained further upstream.

Mr Thompson's and Mr MacDonald's objections to the use of the Manning formula were valid on theoretical grounds, but Mr Chacinski had given the practical reasons for its use.

Mr Wolf entirely supported Mr Dobbie in his remarks about Dr Glasspoole's contribution. Mr Dobbie and he were very grateful to Dr Glasspoole and his staff for the great help they had given. If it had not been properly acknowledged he would gladly do so now.

Dr Glasspoole had referred to the application of the Bruton rainfall figures to the Lyn catchment. One of Mr Dobbie's first suggestions had been that, had the storm of August 1952 occurred farther north and east, it would have resulted in much higher flood discharges. The possibility of this, however, had been rejected, on meteorological grounds (basic orographic pattern), but no such objection appeared to exist to the

application of the Bruton records. It was, of course, very unlikely that a storm centre would exactly coincide with a river catchment, but it was not impossible. The analysis in the present Paper was the basis of the constructional work which was to give Lynmouth complete safety.

Mr Green had been very generous in giving advice in the field, and he had now added a contribution to the discussion which contained essential information on which hydraulic computations could be based.

The Authors welcomed Mr Collinge's interesting contribution which focused attention on the difficulty of estimating the period of concentration of a catchment area whose gathering grounds and rivers had not been surveyed in sufficient detail to permit a full hydraulic analysis.

Mr Kennard's concept of the " average height " of the catchment clearly possessed great possibilities. The Institution Committee on Floods in Relation to Reservoir Practice had had in mind that both the run-off characteristics of a catchment and its liability to receive intense rainfalls influenced the probabilities of the recurrence of heavy floods. Catchments with a great " average height " would necessarily consist of geologically hard strata to have survived as uplands. They were, therefore, likely to be both steep and rocky, and in consequence to possess high run-off coefficients. Also, orographic rainfall was controlled by chains of hills or mountains. From the practical engineer's point of view, there was no advantage in separating those two aspects, but the Authors felt that Mr Kennard's idea was capable of expansion and refinement, which they hoped might be achieved in a separate communication to the Institution.

The Authors entirely agreed with Mr McClean that no amount of deduction after a flood could effectively replace a proper hydrometric survey and record.

Mr Lloyd's data were of the greatest value, and his most revealing hydrographs (*Figs 16*) represented an important elaboration of the views expressed in the Paper.

The conclusions arrived at by Mr B. D. Richards were of great practical importance, and were being carefully considered.

Correspondence

Mr C. H. Archer, referring to Mr Wolf's remarks on rainfall on p. 531, commented that research made at new rain-gauge sites during the past 2¼ years had revealed that rainfall on the high moor was more than was indicated on existing maps of average rainfall. Whereas the annual average probably just exceeded 60 inches on Dunkery, it almost certainly exceeded 70 inches in at least two areas on the western plateau. Longstone Barrow probably had a little less than 70 inches and Chains Barrow considerably more than 70 inches.

It was incorrect to say that there had been only one rain gauge in the Lyn catchment area at the time of the flood. There had been two: one at Longstone Barrow, in the West Lyn catchment; and the other at Lillycombe, in the East Lyn catchment. The latter was read daily, and was a gauge of many years' standing.

Since the flood a third had been added, close to Chains Barrow in the Hoaroak Water catchment (Grid reference 735142). Like the Longstone gauge, the new one could be read only once or twice a month, but comparison with nearby daily gauges made reasonably accurate estimates possible for intermediate falls (the Longstone and Challacombe gauges were only $2\frac{1}{4}$ miles apart). It was therefore fairly certain that 2·75 to 3 inches had fallen at Longstone within 24 hours on 8–9 September, 1952.

Mr Wolf's statement that no reason had been advanced why other parts of Great Britain were free from 9-inch falls revealed a misapprehension. No such assumption was warranted. Mr Archer pointed out that but for his activities—aided by good luck—the 9-inch reading at Longstone Barrow would not have been obtained; it was most unlikely that the still larger, though unorthodox, figure for Simonsbath village would have come to the notice of authority in time to be checked; and the Exmoor storm would presumably have been listed among the 7- to 8-inch falls. The very fact that, though rain was notoriously heavier in the mountains, both the previous readings belonged to the lowlands, suggested that it might not be the 9-inch falls that were deficient so much as adequate rain gauges and observers in the highland areas most likely to receive them. But how that defect could be remedied without prohibitive expense and demands on man-power was difficult to see. Moreover, even in the lowlands, many privately owned gauges were too small to contain a 9-inch fall. Since the foci of thunderstorms tended to be small, it would be seen that good luck was needed for a 9-inch fall to be caught by an instrument adequate to deal with it, with an observer at hand to read it promptly.

Mr Wolf had stated on p. 533 that his walking survey of erosion patterns had suggested lower figures in the vicinity of Simonsbath than were shown in *Fig. 9*. In other words, despite the admittedly ambiguous nature of evidence based on erosion, he seemed to regard his own valuation of "erosion patterns" as evidence of rainfall superior to measurements of water actually caught during the storm, and then checked on the spot by a Scientific Officer of the Meteorological Office. Mr Archer had been present throughout Mr Bleasdale's investigation of the bucket catch at Simonsbath (to which Mr Wolf had presumably referred), and had witnessed the extreme caution and thoroughness of his inquiry into both the technical and human factors involved. The estimate of 10 inches was not open to reasonable doubt except on the ground that it was probably too low. Of course, as Mr Bleasdale had observed, the area of that unprecedented catch was probably very slight—but, again, it was possibly not the only one of its kind.

In assessing the frequency of comparable storms over Exmoor in the

past, it appeared that Mr Wolf had overlooked a very useful flood gauge dating back at least 500 to 700 years, namely, Tarr Steps. That very elaborate clapper bridge, as it had stood until August 1952, had been accurately described by S. H. Burton in his book on Exmoor (pp. 102–103).[28] It had then been swept away in a night—all but one span of 7 feet out of a bridge of seventeen spans. Even if, as some thought, the Steps were medieval, it seemed unlikely that anyone in past times would have gone to the immense labour of rebuilding them if total destruction with considerable dispersion downstream had once occurred. Minor damage without dispersion (as in 1942) was, of course, easily possible and easily repaired. Admittedly the Steps were on the eastern fringe of the Moor, but any major orographic or semi-orographic rain-storm on Exmoor would almost certainly have affected the Barle, which itself rose within $4\frac{1}{4}$ miles of Lynmouth. If, therefore, the storm of 1796 had been of similar intensity, it would surely have been displaced on to lower ground and, like the Cannington storm, it would certainly have been much more local. Such siting was just possible, but it was extravagant to suppose that it would often recur, to the exclusion of storms at nearby sites, any of which would certainly have destroyed Tarr Steps. It was surely more logical to assume that for 500 to 700 years—and perhaps much longer—no equally intense and widespread storm had occurred on Exmoor; but at the same time it should be recognized that the laws of chance and the tide of climatic change might alike cause such storms to be more frequent in the future, as no doubt they had been at times in the more remote geological past.

That evidence drawn from Tarr Steps led to a point of principle which Mr Archer considered important. Had not expert attention been focused too narrowly on Lynmouth and the Lyn river to the neglect of lessons, useful even to Lynmouth itself, that might be drawn from nearby river systems? To revert to Tarr Steps once more, could not some evidence of the forces involved be drawn from the weights of the stones and the distances that they had been carried? For instance, one clapper stone weighing about 1 ton had been carried 157 feet downstream. Another of about $\frac{3}{4}$ ton had been carried 35 feet transversely across the central channel. Others had been carried various distances up to 30 feet. (That information had been kindly supplied by Mr D. A. Alves, Bridge Engineer of the Somerset County Council, under whose supervision the Steps had just been rebuilt. He had stated that a detailed survey had been made between 1930 and 1940 when the County Council had taken over responsibility for the Steps, and that had enabled the individual stones to be identified, and their post-flood positions to be recorded.)

Sir Claude Inglis observed that in order to eliminate the necessity to choose values of n for the various sites for use in Manning's formula, Mr Crump had worked out discharges at the twelve sites listed by the Author,

[28] See ref. 12, p. 546.

on the assumption (justified by much experience) that the discharges at the selected sites would be slightly less than critical for the depths of water, and hence would be unaffected by the slopes. Mr Crump had calculated the discharges, using the Boussinesq critical velocity values and those were given in Table 3 (see p. 554). In that Table were also shown discharges worked out by the Author using the Manning formula based on values of n estimated on site, and also discharges based upon Mr Lacey's final regime formula

$$V = 16(R^{\frac{3}{4}}S^{\frac{1}{2}})$$

Mr Crump, in the Discussion, had correctly stated that that formula had been derived from data with flatter slopes than in the Lyn, and that the formula was, in any case, only correctly applicable to the dominant stage of discharge ; but he had not given any idea of the error likely to occur by applying the formula to maximum-discharge conditions. Table 9 quoted the values obtained by Dr J. K. Malhotra for a boulder river in India,[29] and gave an indication of the order of that error.

TABLE 9

	Q	$S \times 10^3$	P	R	V	C_2
Maximum flood values .	71,371	1·6	408	12·64	13·84	1·35
Minimum flood values .	3,260	0·21	263	5·97	2·08	0·65

The maximum figures were for an ordinary, not a maximum, flood, from which it would appear that for maximum-flood conditions, the coefficient in the Lacey formula should probably be increased by more than the 35 per cent worked out by Dr Malhotra for an ordinary flood ; but that might be largely counterbalanced by the side slopes in the Lyn being steeper than those in the river on which Dr Malhotra carried out his observations. The point to be emphasized was how wrong it would be to use the Lacey final regime formula for estimating a maximum flood ; and such agreement as might exist between the Author's figures and the Lacey final regime formula figures for the Lyn must be largely fortuitous.

Omitting those cases where the slopes were flat enough for the velocities to be less than critical, namely 1, 3, and 4, the comparative flows by the Manning formula with the Authors' assumed values of n, and the Boussinesq discharge formula (Table 10) gave a root mean square value of approximately 0·9.

Generally, the Boussinesq figures confirm the values worked out by the Author ; and had the widths been used instead of the wetted perimeter, the

[29] Annual Report (Technical) of Central Board of Irrigation, 1943.

TABLE 10

	Q_M	Q_B	Q_B/Q_M
2	3,435	4,090	1·19
5	4,810	4,490	0·93
6	10,095	7,625	0·76
7	15,070	12,550	0·83
8	14,595	14,380	0·95
9	3,507	2,670	0·73
10	3,855	2,775	0·72
11	4,554	3,880	0·85
12	8,930	9,467	1·06

differences would have been still less. Taking everything into account the Authors have probably over-estimated some of the discharges.

The Authors had stated that values of n ranging from 0·035 to 0·15 had been quoted in the American National Resources Committee publication "Low Dams"; but the photographs in that publication showed that the high values had been caused, in every case, by vegetation, not rock cuts. In 1941, Sir Claude had had the value of n worked out for the Malegaon Rock Cut of the Nira Left Canal in Bombay Presidency and it was found to be 0·042. In that case the bed was relatively smooth, with n of, say, 0·025, and since the bedwidth was about twice the depth, n of the sides must have been approximately 0·6, which tended to confirm the Authors' evaluation.

With regard to *Fig. 7*, although the run-off from the Lyn catchment was very high for England, it was much less than had been observed for rivers in India. An analysis of the run-off data in the Bombay Deccan[30] showed that an envelope curve which gave a satisfactory fit for the data for maximum discharges was:

$$Q = \frac{7{,}000A'}{\sqrt{A' + 4}}$$ where Q denoted discharge in cusecs, and A' denoted areas in square miles.

Re-written in terms of run-off per thousand acres for areas stated in thousands of acres, that formula became:

$$Q = \frac{10{,}930A}{\sqrt{1{\cdot}563A + 4}} = I$$ where A denoted areas in thousands of acres.

Fig. 17 gave in addition to the data shown in *Fig. 7*, curves for 0·25I,

[30] Bombay P.W.D. Technical Paper No. 30. A Note on a critical study of run-off and floods of Catchments of the Bombay Presidency.

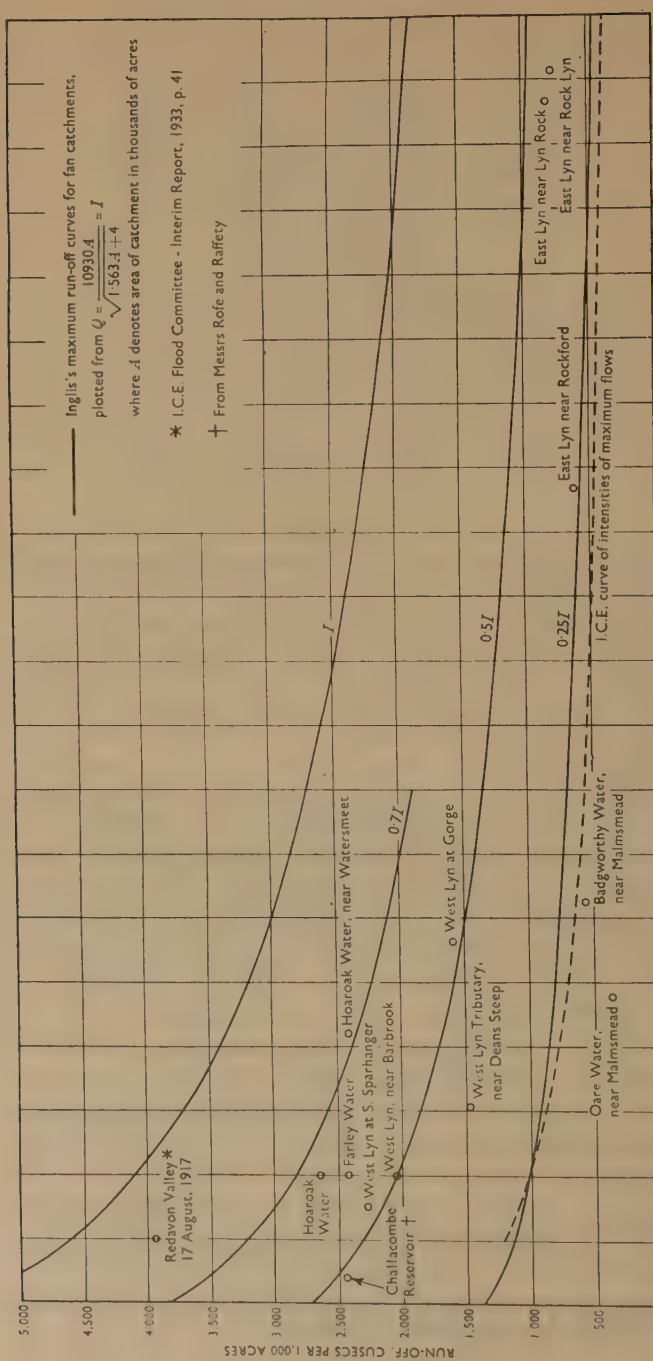
Inglis's maximum run-off curves for fan catchments, plotted from $Q = \frac{10930A}{\sqrt{1563A}+4} = I$
where A denotes area of catchment in thousands of acres
* I.C.E. Flood Committee - Interim Report, 1933, p. 41
† From Messrs Rofe and Raffety
RUN-OFF. CUSECS PER 1,000 ACRES
CATCHMENT AREA. THOUSANDS OF ACRES
Redavon Valley* 17 August, 1917
Hoaroak Water
Farley Water
Challacombe Reservoir †
Hoaroak Water, near Watersmeet
West Lyn at S. Sparhanger
West Lyn, near Barbrook
West Lyn at Gorge
West Lyn Tributary, near Deans Steep
Oare Water, near Malmsmead
Badgworthy Water, near Malmsmead
East Lyn near Rockford
East Lyn near Lyn Rock
East Lyn near Rock Lyn
I
0·77
0·51
0·25I
I.C.E. curve of intensities of maximum flows

Fig. 17

FLOOD-INTENSITY/AREA DIAGRAM

0·5*I*, 0·7*I*, and *I*. It would be seen that 0·25*I* was in close agreement with the " I.C.E. curve of the intensities of normal maximum floods " shown in the 1933 Interim Report of the Institution Committee on Floods; that 0·5*I* agreed with the West Lyn data; that Hoaroak Water data were close to 0·7*I*; and that the Redavon Valley figure was equivalent to 0·84*I*; but with that one exception, nothing had yet been recorded in Britain approximating to *I*. It should not be thought, however, that that was because there were places in the Bombay Presidency with very intense rainfall—up to 300 inches fall in 2½ months. Actually, maximum floods in India tended to be slightly higher in areas where the annual rainfall was less than that in Great Britain.

Lest the Paper should give the impression that large quantities of boulders were swept down the East and West Lyn Rivers during the flood disaster, it was pertinent to state that although large changes might take place in the disposition of boulders in rivers during floods, the rate of progression was very slow. In general, odd boulders became unstable when the bed around them was scoured away, and then rolled a short distance until they settled in a hollow. There they might remain for many years until they were worn down by abrasion or again became unstable due to scour of the finer materials around them. Inspection showed that most of the erosion was of the banks, in restricted lengths. As a result, boulders became exposed, but no evidence could be found of any general bed movement. In the lower part of the West Lyn, where many of the large boulders had been removed, there was likely to be heavy scour until a new regime was set up, exposing a new bed of larger boulders.

A " step " seemed called for at that point, to reduce the excessive gradient.

Mr George Bransby-Williams observed that, since his formula had been referred to, it might be worth mentioning that that formula had originally been used in India to arrive at approximate figures for times of concentration on hilly catchments of comparatively regular shape. It had first been published in Britain in an article on spillways for Indian reservoirs.[31] It was not intended to apply to every kind of catchment. Actually the figures for the Lyn catchments derived from it were not so far wrong as supposed by Mr Wolf. The fall per 100 feet was not that of the stream, but of the surface of the ground, measured along the most direct route to the outfall. The time of concentration—4·5 hours—arrived at by Mr Collinge, using that formula, seemed to be approximately correct.

Mr Bransby-Williams had one criticism to make of Mr Wolf's dictum that " when designing river control works, it is desirable to know the probable recurrence intervals of various intensities of floods ": he felt that " essential " should be substituted for the word " desirable." All would

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[31] G. Bransby-Williams, " Flood Discharge and the Dimensions of Spillways in India," *Engineer*, vol. 134, p. 321 (29 Sept. 1922).

agree that such information could be most accurately derived from systematic river-gauging records covering long periods (accompanied by automatic rain-gauge records). The difficulty arose in places where (as on the Lyn catchment) no such records existed. The Paper would be studied with the expectation of learning how it was intended to overcome that difficulty in the present case. Actually, it was not clear how the model-investigation was to supply sufficient data for the design of flood prevention works until it had been continued for a long time.

In the United States, there was a fairly wide consensus of opinion that problems of flood flows from catchments were best studied through the medium of unit hydrographs. Mr Bransby-Williams had developed a " basic hydrograph " which was the discharge hydrograph of 1 inch of " effective " rain, falling uniformly over the catchment at a uniform rate throughout the time of concentration. In a recent article,[32] he had described a method of constructing a synthetic hydrograph for a catchment for which no stream gaugings had been made. Briefly, it consisted in dividing the catchment into zones and calculating the rates at which the discharges arrived at the outfall in successive intervals of time. The rising arm of the hydrograph—the " concentration curve "—was the summation curve of the rates of discharge from the zones. The rain was assumed to cease at the end of the time of concentration. The descending arm, or " storage curve," was calculated from a modification of R. T. Zoch's formula :[33]

$$C_1 - t = C_2 \log Q$$

where t denoted the time from the beginning of the storm in terms of the time of concentration (taken as unity) ; Q the percentage of the peak rate of discharge ; C_1 and C_2 coefficients appropriate to the character of the catchment. The storage curve was slightly flattened at the summit.

The area under the hydrograph represented the total effective rain. That enabled the peak rate of discharge to be computed.

If the theory was accepted that, on any catchment, run-offs of single storms of a given duration produced hydrographs of similar shape and with the same base length, it followed that the peak rates of run-off in storms on the catchment that continued throughout the time of concentration would be in proportion to the rates of effective rainfall producing them.

If it was also assumed that the maximum rate of run-off from a catchment was that caused by the maximum intensity of effective rainfall that could continue throughout the time of concentration, the probable frequency of any peak rate of flood discharge could be arrived at if the probable frequency of the maximum effective rainfall intensities that could continue for that time could be established.

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[32] G. Bransby-Williams, " Flood Intensities and Frequencies on British Catchments," *Engineer*, vol. 194, p. 247 (22 Aug. 1952).

[33] R. T. Zoch, " On the relation between rainfall and stream flow," *Monthly Weather Review* (Washington, U.S.A.), vol. 62 (1934), p. 315.

The only source of data for rainfall intensity/duration of fall ratios in Great Britain that Mr Bransby-Williams had available was "British Rainfall." In that publication only heavy falls in short periods, and heavy daily rainfalls were recorded. Moreover, it was to be supposed that some of the heaviest falls were not recorded at all. Confirmation of that supposition was to be found in the Paper; for, if it had not been for the gauge on Longstone Barrow, and Mr Archer's action, there would have been nothing to show that anything like 9 inches had fallen in 24 hours.

On the basis of such information as he had been able to collect, Mr Bransby-Williams suggested the following formula for the relationship between intensity of rainfall, duration of fall, and probable frequency at any station, as suitable for parts of Britain (including the west of England) for storms lasting from 1 to 24 hours:

$$i = \frac{1 + \log \frac{N+6}{20}}{t^{0.8}}$$

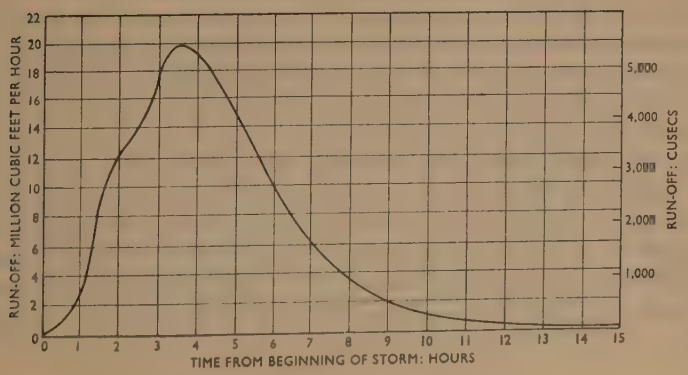
where i denoted intensity of rainfall in inches per hour; t denoted duration of fall in hours; and N denoted probable frequency in years.

It might be noted that that formula would make the frequency of the I.C.E. Committee's "normal maximum rainfall" rates something more than 200 years and that of the "catastrophic" rainfall about 50,000 years.

Messrs Bleasdale and Douglas had calculated that, during the Lynmouth storm, 56 per cent of the day's rain on Exmoor had fallen in 4 hours. Assuming the same proportion on Longstone Barrow, approximately 5 inches would have fallen there in 4 hours at the rate of 1·25 inch per hour. According to the above formula, that was a rainfall intensity that would be expected to occur once in 12,000 years.

Fig. 18 was the basic hydrograph for the Lyn catchment, constructed from the information in the Paper. The time of concentration was taken

Fig. 18



to be 3·5 hours. One inch of rain in 3·5 hours was equivalent to 0·287 inch per hour. The peak rate of run-off shown in the hydrograph was about 5,600 cusecs.

The problem of calculating the probable frequencies of peak flood discharges resolved itself into an estimate of the proportion of the rainfall at the storm centre that was likely to represent the effective rainfall on the catchment in any particular case. There one had to enter the realm of surmise ; but supposing 80 per cent of the rainfall rates derived from the formula became effective on the Lyn catchments, the frequencies of floods of the respective intensities would be those shown in Table 11.

TABLE 11

Probable frequency : years	Peak discharge : cusecs	Cusecs per sq. mile
10	5,700	146
100	9,900	253
200	11,600	297
1,000	15,500	397
5,000	19,500	498
20,000	23,000	587
50,000	25,200	642

The figure for the rate of run-off obtained from the " normal maximum " curve of the I.C.E. Floods Committee was 214 cusecs per square mile.

On the same basis of calculation a rainfall of 1·25 inch per hour for 3½ hours at Longstone Barrow represented a peak flood intensity of 19,600 cusecs on the Lyn catchment, compared with Mr Wolf's 25,000.

The foregoing figures indicated that the Lynmouth flood had been so abnormal that the recurrence of a flood of similar intensity within any period of time to which it was necessary to look forward was extremely improbable, and that in any control works projected it would be necessary only to provide for a considerably lower rate of run-off.

Dr W. G. V. Balchin considered it should be placed on record that although not mentioned or discussed by the Authors, a logical geomorphological solution to the problem of periodic flooding in Lynmouth did exist. Numerous cases were known throughout the south-west peninsula of river diversions as a result of marine erosion, and in time to come the East Lyn would be captured once again by the sea to the south of Wind Hill. The same result could be prematurely achieved by the construction of a check dam on the East Lyn below Watersmeet and the diversion of the river through Wind Hill by a tunnel about 900 yards long. If desired, a miniature " Hoover Dam " arrangement with an overspill tunnel could ensure that the normal East Lyn continued to flow in its present channel but that

flood-waters found a direct outlet onto Sillery Sands without passing through Lynmouth.

His second point related to the wider problem of river flooding. Great Britain generally was perhaps not so free from intense falls of rain as the remarks by Mr Wolf might suggest and the Lynmouth tragedy had once again brought to the fore a problem which had not yet been seriously faced.

River flooding occurred in two ways: it could be either periodic or episodic. There could be fairly regular flooding related to a repetitive regime in the life of the river. Winter rains or spring snow melt could produce annual floods, in which case the local inhabitants were fully aware of the need to adapt their way of life to that of the river. Flood-plain sites were avoided when buildings were erected, and bridges were adapted to maximum known flood conditions. The flood plain of a maturely developed river might, however, be subject to longer-term inundations, and those had frequently caught unwary house-builders and industrial operators. For example, in the Thames valley.

A second type of flooding, exemplified by the Lynmouth disaster, could be regarded as episodic and was related to abnormally heavy storms. The effect of exceptional precipitation might be further intensified by immaturely dissected topography where water was concentrated in narrow valleys with torrential tracts. That could have devastating effects. Flooding of that nature might affect an area only at infrequent intervals and any one incident might pass out of living memory before the next was experienced. The local inhabitants therefore might be lulled into a false sense of security as memories of the last serious flood faded into the past.

But over Britain as a whole, excessive rain which could lead to that type of flooding occurred with a frequency probably much greater than was generally realized. A fall of 3 inches of rain in one day was regarded as quite exceptional, and yet in British rainfall records one found dozens of cases listed from 1865 onwards, with falls of up to 9½ inches as a possibility. Some of those falls had produced a series of historic floods, the most notable of which included the Moray disaster of August 1829; the Bodmin Moor episode of 16th July, 1847, when all the bridges but three on the Camel and Inney Rivers had been swept away; the Norwich floods of 25th August, 1912, which had caused great damage to the city; the River Lud (Lincolnshire) disaster of 29th May, 1920, when twenty-three fatalities had occurred and 1,250 people had been rendered homeless; and the Vale of Conway floods of November 1925, when sixteen people had been drowned. More recently there had been the Afon Mawddach floods of May 1944; the south-east Scotland floods of 12th August, 1948, which had resulted in the loss of many bridges, much cattle, and extensive damage to property; and finally the Exmoor disaster of 15th August, 1952, with twenty-eight deaths, and the loss of much of Lynmouth and seventeen bridges.

Those were only some of the more notable episodes in a long and

continual story of flooding. The pressure of an increasing population upon available land space was already acute in the British Isles, the number of towns and villages in valley situations was uncomfortably large, and already many potentially dangerous flood sites had certainly been built upon. Without guidance, more would follow, and without precautionary measures it could only be a matter of time before another disaster similar to that at Lynmouth overtook an unsuspecting community. In many areas, future building activity, bridge construction, rail and road placements, etc. should be clearly related to the extreme river flood conditions, both periodic and episodic, likely to be experienced in the locality. A systematic and serious study of the problem seemed to be called for, and a synthesis of information was needed from the meteorological records of the past century, earlier historical evidence, and also the longer-term geomorphological record: from that data the hydraulic engineer could distinguish potentially dangerous areas and recommend future precautionary measures.

Mr William Allard suggested that a useful comparison with the values of the coefficient n adopted by Mr Dobbie would be found in the Appendix of Mr G. N. Croker's Paper on the records of flows in the River Wye system,[34] since n was there calculated from measured discharges.

As to what Dr Glasspoole had said about the recording of future rainstorms, much of that applied in principle also to the measurement of future river discharges. It was sometimes asserted by champions of the Inland Water Survey that, if it continued long enough, it would provide measurements of the extreme flows, high or low, of every stream in Great Britain. Such a generosity of performance was surely neither practicable nor necessary.

The object of the Survey was rather to secure comprehensive and continuous measurement of river flows at a limited number of stations arranged in a systematic network to give representative results for the whole of Great Britain. From what happened at those stations much could be deduced as to what had happened elsewhere without being measured. General laws of river behaviour also should be ascertainable from the data collected and so lessen the need for direct measurement. Any additional short-term measurements at other stations would, however, have their significance much increased by what was available from the network of permanent or long-term stations.

Just as the existence of a supply of reliable Ordnance Survey map-sheets saved other land-surveyors and engineers an enormous amount of labour, although it did not always relieve them entirely of the need to survey for themselves, so the Inland Water Survey was an instrument of economy and not of extravagance in the spending of effort upon river flow measurement, and as such was greatly in keeping with the general need for truly productive expenditure of labour in the post-war years. Planned economy of skilled effort in the gauging of both rainfall and run-off was,

[34] J. Instn Wat. Engrs, vol V, No. 1, p. 39 (Feb. 1951).

moreover, something that made whatever results were so obtained far more valuable than those of a greater mass of unsystematic measurements of possibly doubtful accuracy.

Naturally, once the Survey had working for it a number of experienced river gaugers, they might be expected to make special efforts to measure abnormal flows at unusual stations, just as Mr Archer had acted specially in the case of the Lynmouth rainfall. Again, on such occasions, methods of measurement other than the usual ones might well be the only practicable means of estimating what had occurred, as had been well demonstrated in the Paper by both Authors and the staff of the Meteorological Office, by, for example, the West Lyn scale model and observations of erosion, deposition, and boulder movement. In like manner, river velocity measurements at one-fifth depth only might be used at times, following a suggestion of Mr G. N. Croker in his Paper already mentioned.

Mr Dobbie, in reply, thought that Sir Claude Inglis had minimized the boulder movement in his contribution and the Authors supported in preference the version of Mr G. W. Green, who had examined the river systems, after the floods, on behalf of the Geological Survey, and whose findings agreed with their own observations. For example, one heap of boulders left standing against the Lyndale Hotel had contained 6,000 cubic yards. Mr Archer had given some very useful figures for Tarr Steps, where the River Barle was not so precipitous as the Lyn at Lynmouth.

Dr Balchin's solution of a tunnel had not been overlooked. The difficulty was that the river walls in Lynmouth would have to be rebuilt in any case to afford stability to exposed buildings and to accommodate the West Lyn, which joined the valley below the tunnel site. The additional cost of a wider channel would not be proportionate, so the channel was not economic. Dr Balchin, as a wise geomorphologist, had included a check dam.

Mr Allard, an equal enthusiast in his sphere of river gauging as Dr Glasspoole in that of rainfall measurement, had a different task. The apparatus required was more complicated and the proficient amateurs were very few. Authorities, including some river boards, had been slow to realize the essential benefits of selective intelligent gauging. In the Authors' very strongly held opinion, that was a matter in which no country could afford to economize, least of all Great Britain, with her dense population and ever-expanding agriculture and industry.

The appendix to Mr G. N. Croker's Paper showed values of n ranging from 0·022 to 0·140, calculated back from gaugings. When an experienced engineer selected his n he would not only see the roughness of the site but would have in his mind, consciously or subconsciously, such calculations for previous sites examined. The more such experience was built up, the greater would be the accuracy of hydraulic data of floods, since it would be too much to expect them all to pass through observed gauging stations. That appraisal of the situation perhaps answered Mr R. H. MacDonald's

suggestion that the hydraulic radius R was more difficult to determine than n.

Mr Wolf, who also replied, felt that Mr Archer's vigorous contribution contained much useful additional information, and also some definite assertions on matters about which, perhaps, sufficient evidence was not yet available. The remarks on "a Scientific Officer of the Meteorological Office" attributed to Mr Wolf views which he did not hold and then proceeded to contradict them! No question had been raised about the meteorological facts as established by direct observation. Where, on the other hand, rainfall had to be derived from erosion and flow patterns, the geologist and the hydraulic engineer would be more competent to judge, and on that point Mr Wolf was quite unrepentant. For confirmation he referred Mr Archer to several contributions to the discussion. If the estimate of 10 inches per day at Simonsbath was too low, the evidence of erosion patterns and bed-load transport (which were a useful basis for comparison) indicated that the rainfall at the head of the Hoaroak valley might have approached the figure of 12 inches which Mr Richards had used as the maximum for the British Isles.

Mr Wolf gladly accepted the corrections proposed by Mr Bransby-Williams. The purpose of the model, so far as the river-gauging was concerned, was to provide a stage-discharge curve. If the water level at the gauge (or, preferably, in a stilling well) were recorded over a long period, the corresponding flows through the gauging section could be derived without any further calibration, and a full picture of the behaviour of the West Lyn at Lynmouth could be deduced from them. The model was not the means of avoiding regular water-level observations or continuous recordings.

Whilst they did not wish to question the formula for the probable frequency of occurrence of storms (N) as given by the formula on p. 583. the Authors thought that its application to the Lyn catchment resulted in longer probable recurrence intervals of floods (column 1, Table 11) than the evidence in the Paper and in the discussion indicated. With regard to the figure of 25,000 cusecs at Lynmouth, the surveys did not justify a reduction to 19,600. Apart from the model-test, an independent (though indirect) check on the accuracy of computed run-off figures had been provided by Messrs T. M. Chacinski and W. Harris, who had carried out measurements on the Pinkworthy reservoir south of the source of the West Lyn. The peak flow into that reservoir could be calculated from the maximum water level reached, and it was about 9 per cent less than the value corresponding to the discharge quoted in Table 1.

The contribution by Mr Allard, who was in charge of the Inland Water Survey, was appreciated the more because the work of that Survey was of such great importance.

Correspondence on the foregoing Paper is now closed and no further contributions will be accepted.—SEC. I.C.E.

Paper No. 5889

"The Design of Simply Supported Prestressed Concrete Beams for Working Loads"

by

Frederick William Gifford, B.Sc.(Eng.), Ph.D.

(*Ordered by the Council to be published with written discussion.*) †

SYNOPSIS

On the assumption that, at any given time, concrete stresses are within the linear stress/strain range, equations are developed for a prestressed section subject to an external bending moment, the equations being of a form such that the moment carried and steel force are functions of the extreme fibre stresses and the net concrete section.

Limiting conditions of creep and shrinkage are considered in conjunction with the maximum and minimum moments on the section, and two extreme conditions are derived for concrete stresses.

Specifying limiting ranges for the concrete stresses, and introducing factors to deal with self weight and other sustained loads, the equations are reduced to two simultaneous equations to be solved for the depth of the section, for given section proportions. To assist in selecting sections, a general section is broken down into three rectangles approximating to the section, and a Table of section properties is given covering a wide range of sections. Using the tabulated properties, it is shown how a satisfactory section may be rapidly obtained, and it is clearly indicated how poor sections can be improved.

Checking equations are subsequently derived to enable the completion of the design once the section has been fixed.

Similar equations have been derived for beams subject to conversion in handling.

IT is assumed that the simple theory of bending applies (stress/strain relationships are linear), and that no cracking occurs in the concrete.

LIST OF SYMBOLS

(Referring to *Figs 1* and *2*)

D denotes the overall depth of the concrete section.
A_c ,, the area of the net concrete section (n.c.s.).
A_s ,, the area of the prestressing steel.
B ,, the effective width of the n.c.s., defined by $B = \frac{A_c}{D}$.

† Correspondence on this Paper should be received at the Institution by the 15th April, 1954, and will be published in Part III of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

$I_c = \frac{1}{K} BD^3$ is the second moment of area of the n.c.s. about its centroid.

K is a shape factor of the order of 6 to 12.

$p = \frac{A_s}{A_c}$, the proportion of steel.

jD denotes depth of centroid of n.c.s. below top fibres.
fD ,, distance of centroid of prestressing steel from bottom fibres.
M ,, externally applied moment, positive when producing compression in the top fibres.
t ,, stress in the prestressing steel (tension positive).
c ,, stress in the concrete (compression positive).

In order to locate the position of the concrete stress in the depth of the section, the following are used as first suffixes to c :

t to denote top fibres.
b ,, bottom fibres.
c ,, centroid of the net concrete section.
s ,, centroid of the prestressing steel.

In order to specify the particular loading condition, p, X, L, and S are used as second suffixes to c, and as first suffixes to t, with the following connotation :

p : prestressing alone
X: prestressing plus any external moment M_X
L : prestressing plus full working load moment M_L
S : prestressing plus sustained load moment acting at time of transfer of prestress to concrete, M_S.

Theory

It will be noticed that the net concrete section (n.c.s.) is used throughout, this normally being the actual concrete section with due allowance for cavities and areas occupied by prestressing steel, where of account. The exception occurs when unprestressed steel is present, and this can be allowed for by including its equivalent concrete area in the net concrete section.

Prestress Alone

Referring to *Figs. 1* (*a*) and *1* (*b*), the reaction of the steel force $A_s t_p$ acts as an eccentric load on the n.c.s., hence the following may be derived :

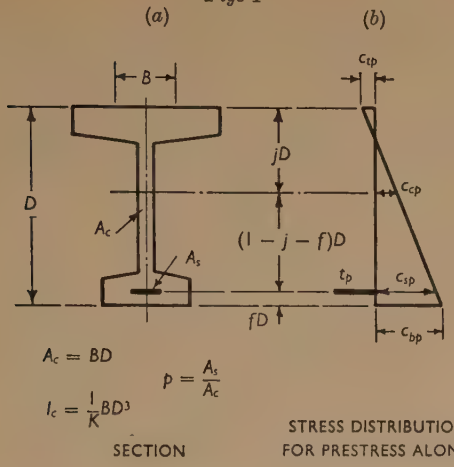
$$c_{cp} = pt_p \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$c_{bp} = pt_p\{1 + (1 - j)(1 - j - f)K\} \quad . \quad . \quad . \quad . \quad (2)$$

$$c_{tp} = pt_p\{1 - j(1 - j - f)K\} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$c_{sp} = pt_p\{1 + (1 - j - f)^2 K\} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

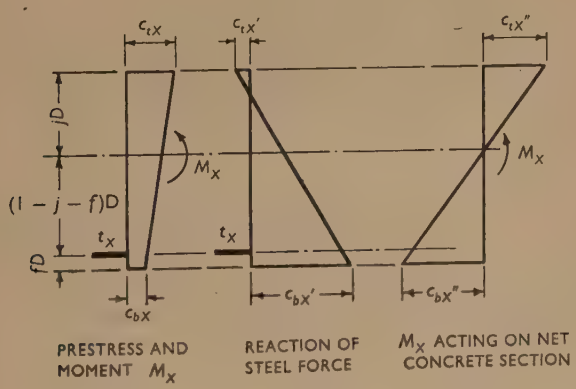
Figs 1



Prestress plus any External Moment M_X

The total stress distribution is shown in *Fig. 2 (a)*. Considering the forces and moments applied to the net concrete section, there are two components —the reaction of the steel force acting as an eccentric compression,

Figs 2



and the external moment. *Figs 2 (b)* and *2 (c)* show the stress components corresponding to these, the notation to indicate the component stresses being the suffixes ′ and ″ respectively. It can clearly be seen that :

$$c_{tX} = c_{tX}' + c_{tX}'' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$c_{bX} = c_{bX}' + c_{bX}'' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

By analogy with (2) and (3):

$$c_{tX}' = pt_X\{1 - j(1 - j - f)K\} \quad . \quad . \quad . \quad . \quad . \quad (7)$$

$$c_{bX}' = pt_X\{1 + (1 - j)(1 - j - f)K\} \quad . \quad . \quad . \quad (8)$$

From the usual $\frac{M}{I} = \frac{f}{y}$ formula, it can be shown that:

$$c_{tX}'' = \frac{jKM_X}{BD^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$c_{bX}'' = \frac{-(1 - j)KM_X}{BD^2} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Combining (5) to (10), and eliminating component stresses, it is seen that:

$$\frac{M_X}{BD^2} = \frac{(c_{tX} - c_{bX})}{K} + (1 - j - f)pt_X \quad . \quad . \quad (11)$$

By the geometry of *Fig. 2 (a)*, and considering horizontal forces:

$$pt_X = c_{tX} - j(c_{tX} - c_{bX}) \quad . \quad . \quad . \quad . \quad . \quad (12)$$

By (11) and (12):

$$\frac{M_X}{BD^2} = \frac{(c_{tX} - c_{bX})}{K} + (1 - j - f)\{c_{tX} - j(c_{tX} - c_{bX})\} \quad . \quad (13)$$

Inspection of equation (13) shows that if the concrete stresses are specified for a given concrete section and location of steel, then the external moment resisted is fixed; further, equation (12) indicates that the steel *force* is also fixed. The rational approach to design is evidently to specify stresses in the concrete, find a section to give these stresses under the applied moments, and then to put in the steel force to suit. The steel stress may be assumed arbitrarily, or preferably calculated to give a specified factor of safety.

Equations (11) and (12) have been derived for a general external moment M_X. The maximum and minimum external moments to be resisted by the section will now be taken into account. These will be denoted by M_L (maximum working load moment) and M_S (sustained moment resisted at time of transfer of stress to concrete).

It is clear that the corresponding equations for M_L are:

$$\frac{M_L}{BD^2} = \frac{(c_{tL} - c_{bL})}{K} + (1 - j - f)pt_L \quad . \quad . \quad . \quad (14)$$

$$pt_L = c_{tL} - j(c_{tL} - c_{bL}) \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

When $c_{bL} = 0$, it can be seen that:

$$pt_L = c_{tL}(1 - j) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

$$\frac{M_L}{BD^2} = c_{tL}\left\{\frac{1}{K} + (1 - j)(1 - j - f)\right\} \quad . \quad . \quad . \quad (17)$$

In addition to two limiting conditions of external moment, two limiting conditions of creep and shrinkage must also be taken into consideration. These are :—

(*a*) Full creep and shrinkage.

(*b*) Instantaneous stresses—that is, stresses present under the condition of creep and shrinkage which occur at the time of release of stress on to the concrete.

If the two loading conditions are combined with the two limiting conditions for creep and shrinkage, four conditions of stress are obtained in effect ; it will be shown that two of these four conditions are generally the extreme cases. Referring to *Figs 3*, which numerically illustrate the various stress changes producing the four conditions of stress, the stress diagrams (*a*) to (*e*) represent instantaneous stresses ; (*f*), (*g*), and (*h*), stress changes caused by the creep of concrete and steel, and the shrinkage of the concrete (which decrease the steel stress) ; and (*i*) to (*m*) represent conditions of full creep and shrinkage. In *Figs 3* :

(*a*) and (*i*) represent prestress alone.

(*b*) is the same as (*j*) and both represent stress changes due to M_S.

(*c*) is the sum of (*a*) and (*b*).

(*k*) is the sum of (*i*) and (*j*).

(*d*) and (*l*) are the same, and represent stress changes due to $M_L - M_S$, the diagrams being proportional to (*b*) and (*j*).

(*f*), (*g*), (*h*) are the same, and represent stress changes caused by reduction of steel stress accompanying creep and shrinkage, the diagrams being proportional to the sum of (*a*) and (*i*).

(*e*) represents the sum of (*c*) and (*d*) ; the sum of (*a*) and (*f*) is (*i*) ; the sum of (*c*) and (*g*) is (*k*) ; the sum of (*e*) and (*h*) is (*m*), which is also the sum of (*k*) and (*l*).

Inspection shows that the worst concrete stresses correspond to :

(1) M_L acting with full creep and shrinkage ;

(2) M_S acting with instantaneous stresses.

It will be realized that when the prestress alone produces compression in the upper fibres, the worst top-fibre stresses correspond to the other two cases ; since such cases are exceptional, and since the variation of top-fibre stress caused by creep and shrinkage is small, this case can be neglected in developing a method of selecting a section.

In order to differentiate between the two conditions of creep, the notation for stresses given so far is reserved for the condition of full creep and shrinkage, and to indicate the condition of instantaneous stresses, a prefix $'$ is used. Thus for M_S acting, and instantaneous stresses, the equations corresponding to (11) and (12) are :

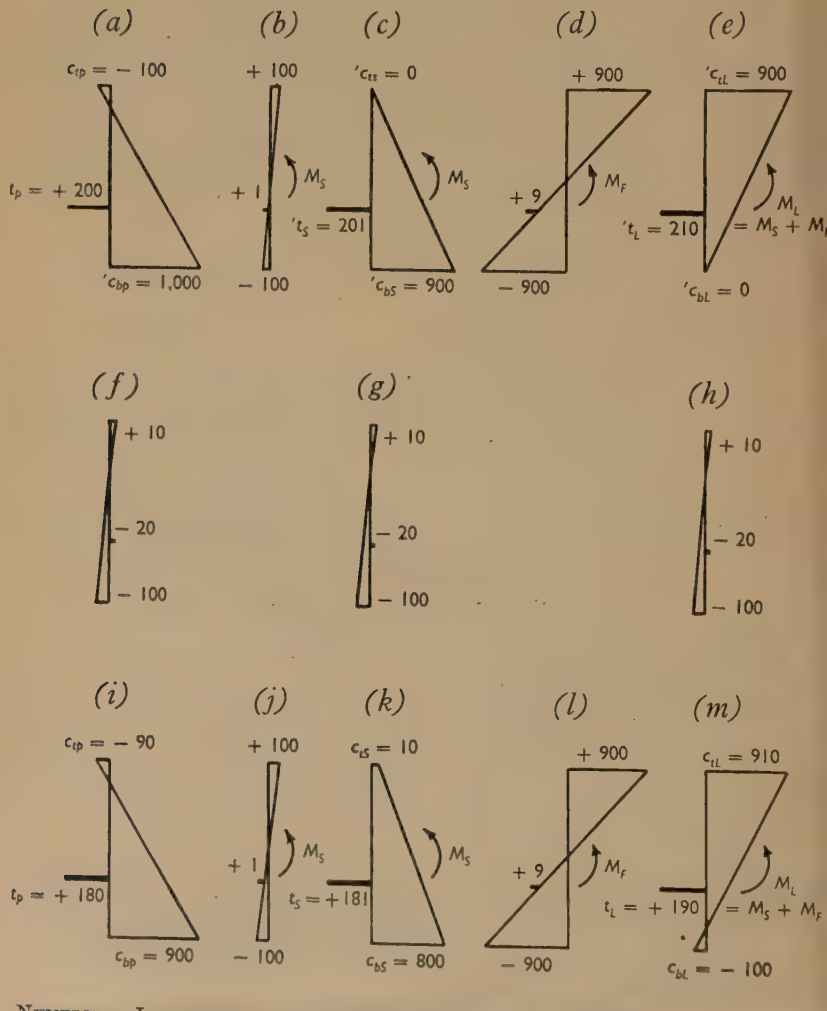
$$\frac{M_s}{BD^2} = \frac{('c_{tS} - 'c_{bS})}{K} + (1 - j - f)p.'t_S \quad . \quad .' \quad (18)$$

$$p \cdot {}'t_S = {}'c_{tS} - j({}'c_{tS} - {}'c_{bS}) \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

It has been shown mathematically, and substantiated by tests, that the change of steel stress from ${}'t_S$ to t_L is small, although it depends on a large number of factors. It is convenient in design to assume a safe limiting value of λ in :

$${}'t_S = (1 + \lambda)t_L \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Figs 3



NUMERICAL ILLUSTRATION OF THE STRESS CHANGES DUE TO EXTERNAL MOMENT, CREEP, AND SHRINKAGE

λ has a range of about 0 to 0·15, the latter being the safe limiting value for normal stresses.

By definition, c_a is the allowable concrete compressive stress at 28 days;

δc_a is the allowable concrete compressive stress at release of stress on to the concrete;

and $M_F = M_L - M_S$. M_F is a known design value, whereas M_S and M_L are not, except where self-weight is negligible.

For the purpose of developing equations for direct selection and checking of sections, the following values of extreme fibre stresses may be assumed, in terms of allowable stresses:

$$c_{tL} = \rho c_a \qquad c_{bL} = 0 \qquad 'c_{bS} = \theta \delta c_a \quad . \quad . \quad . \quad . \quad (21)$$

$'c_{tS}$ cannot be specified since under M_L the centroidal concrete stress is $(1-j)\rho c_a$, and since λ is specified the centroidal stress under M_S is $(1+\lambda)(1-j)\rho c_a$. With $'c_{bS} = \theta \delta c_a$, clearly the value of $'c_{tS}$ is fixed. Obviously, $'cts$ should not be less than zero (or an allowable tension).

The coefficients ρ and θ may be taken as unity when dealing with the selection of a section; for checking a given section it will only be necessary to prove that their values do not exceed unity in order to show that stresses are within the allowable range.

Substituting in equations (16), (17), (18), and (19), using (20) and (21):

$$pt_L = \rho c_a (1-j) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

$$\frac{M_L}{BD^2} = \rho c_a \left\{ \frac{1}{K} + (1-j)(1-j-f) \right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

$$p.'t_S = (1+\lambda) pt_L = 'c_{tS}(1-j) + \theta \delta j c_a \quad . \quad . \quad . \quad . \quad . \quad (24)$$

$$\frac{M_S}{BD^2} = \frac{('c_{tS} - \theta \delta c_a)}{K} + (1-j-f)(1+\lambda)\rho c_a (1-j) \quad . \quad (25)$$

By (22) and (24):

$$'c_{tS} = c_a \left\{ (1+\lambda)\rho - \frac{\theta \delta j}{(1-j)} \right\} \quad . \quad . \quad . \quad (26)$$

By subtracting (25) from (23) and eliminating $'c_{tS}$ by (26), and using M_F as defined, the following is obtained:

$$\frac{M_F}{c_a BD^2} = \frac{\theta \delta}{K(1-j)} - \rho\lambda \left\{ \frac{1}{K} + (1-j)(1-j-f) \right\} \quad . \quad . \quad . \quad . \quad (27)$$

Inspection of equation (27) shows that if the section proportions are specified—that is, K, j, f, and $\frac{B}{D}$ — this equation may be solved for D for given c_a, δc_a, λ, and M_F, the coefficients ρ and θ being taken as unity. It is, however, independent of M_S, and a further equation can be developed with which equation (27) is simultaneous.

Let M_G denote self-weight moment
M_A denote any additional sustained load moment carried at the time of transfer of stress to the concrete :

$$M_A = \phi M_F \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

where ϕ is a known design value.

$$M_S = M_G + M_A = M_L - M_F \quad . \quad . \quad . \quad (29)$$

For a simply supported beam of constant weight per unit length it can be shown that :

$$M_G = 1{\cdot}56BDL^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

assuming a density of concrete of 150 lb. per cubic foot, all units being in lb. and inches, except the span L, which is in feet. Thus by (28) and (29) :

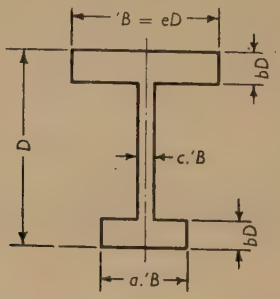
$$\frac{M_L}{c_a BD^2} = \frac{1{\cdot}56L^2}{c_a D} + \frac{(1+\phi)M_F}{c_a BD^2} \quad . \quad . \quad . \quad (31)$$

Substituting by equations (23) and (27), it will be found that :

$$\frac{1{\cdot}56L^2}{c_a D} = \rho\left\{\frac{1}{K} + (1-j-f)(1-j)\right\}\left\{1+\lambda(1+\phi)\right\} - \left\{\frac{(1+\phi)\theta\delta}{K(1-j)}\right\} \quad (32)$$

Equation (32) is simultaneous with (27).

Fig. 4



FLANGED SECTION COMPOSED OF RECTANGLES

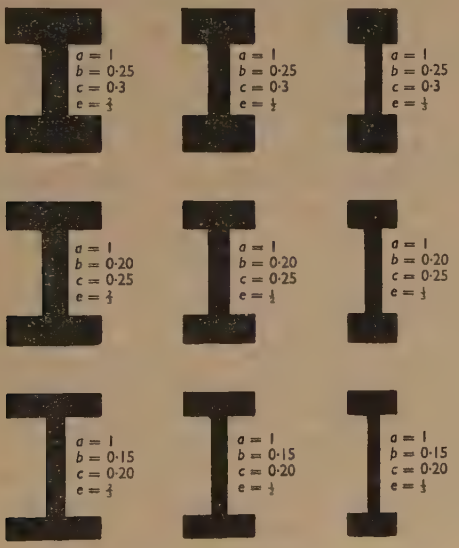
To expedite the selection of a section, where this is not rectangular, it is convenient to use a flanged section built up of three rectangles, as shown in *Fig. 4*. Depth is D, top flange width is $'B = eD$, bottom flange width $= a\,.\,'B$, flange thicknesses $= bD$ and web thickness $= c\,.\,'B$. Thus if specific values are chosen for the proportion ratios a, b, c, and e, the values of K, j, and $\frac{B}{D}$ may be calculated ; K and j are independent of e, of which $\frac{B}{D}$ is a linear function. Table 1 gives the values of these coefficients for the normal range of sections required, the accuracy being such that linear interpolation of intermediate values are at least as accurate as values calculated on a 10-inch slide rule.

Method of Selection of Section

(*a*) *Flanged Section, Flange Width not Restricted*

It is convenient first to assume that $a = 1$. Reasonable values of b, c, and e, are selected according to the type of beam required: for small beams, say, $b = 0{\cdot}2$ to $0{\cdot}25$ and $c = 0{\cdot}25$ to $0{\cdot}4$; for large beams, say, $b = 0{\cdot}1$ to

Figs 5



Range of Sections drawn to Scale

$0{\cdot}2$ and $c = 0{\cdot}1$ to $0{\cdot}25$. *Fig. 5*, which shows a range of sections to scale, should be of assistance. The values of K, j, and $\frac{B}{D}$ are obtained from Table 1. Equations (27) and (32), with ρ and θ unity, become :

$$\frac{M_F}{c_a B D_F{}^2} = \frac{\delta}{K(1-j)} - \lambda\left\{\frac{1}{K} + (1-j)(1-j-f)\right\} \quad . \quad . \quad . \quad . \quad . \quad (33)$$

$$\frac{1{\cdot}56L^2}{c_a D_S} = \left\{\frac{1}{K} + (1-j-f)(1-j)\right\}\left\{1 + \lambda(1+\phi)\right\} - \left\{\frac{(1+\phi)\delta}{K(1-j)}\right\} \quad (34)$$

These may now be solved for D_F and D_S (which are merely the values of D as obtained from the two equations), as functions of f, since all other terms are known design values. Equations are obtained, of the form :

$$D_F{}^3 = \frac{w}{(x+f)} \quad \text{and} \quad D_S = \frac{y}{(z-f)} \quad . \quad . \quad . \quad . \quad (35)$$

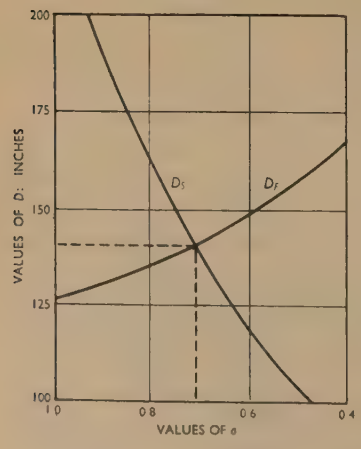
where w, x, y, and z are numerical values.

A reasonable minimum value of f is now chosen—say, 0·2 to 0·3 for small beams, 0·1 to 0·2 for large beams—and the values of D_F and D_S are calculated. If they are equal, it means that the section proportion and f-value chosen are such that, with the D-value obtained, the moments applied will produce the specified stresses (see equations (21)). If they are not equal, the proportions are unsatisfactory, and can be improved. The two main factors which affect the D-values are a and f, and it may be readily shown that D_S is decreased and D_F is increased by decreasing a or f, and vice versa. Thus if D_S is greater than D_F the section can be improved by decreasing a, but f cannot be decreased since its minimum value has been used. By choosing two other values of a, obtaining the section factors from Table 1, the corresponding D_F- and D_S-values may be evaluated, and D roughly plotted against a to find the a-value at the intersection. The corresponding D-value is that required.

If D_S is less than D_F, the section can be improved by increasing a or f. Increasing a implies a bottom flange heavier than the top, and can be done up to a limiting maximum value of j (see later), and is in effect the most economical; the process being, as before, to choose two larger a-values, and to plot D against a, the intersection giving the required D- and a-values. If it is desired not to increase a above unity, then using $a = 1$, two larger f-values are chosen and substituted in equations (35), and D is likewise plotted against f. The intersection gives the required D-value, the corresponding f-value not being the most economical—as will be explained later. Typical $D : f$ and $D : a$ curves are shown in *Figs 6* (*a*) and *6* (*b*). It will be found that, for $D : f$, D_F is approximately linear and has little variation, and in consequence two pairs of points about the intersection are sufficiently accurate to give the D-value.

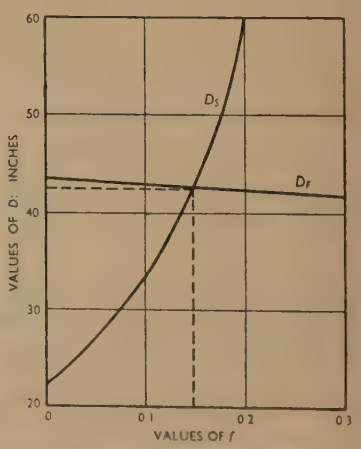
Figs 6

(*a*)



TYPICAL $D : a$ CURVES

(*b*)



TYPICAL $D : f$ CURVES

Having obtained $D_S = D_F$, it is necessary to check that the original assumptions with regard to proportions give a reasonable section from practical considerations, and if they do not, the process must be repeated, intelligently using the information obtained. When the section is satisfactory, it is necessary to convert into practical dimensions, and the following procedure avoids weakening the section in any way.

A practical value for D is selected, not less than theoretical value. The total area of theoretical section is calculated and this is used for the practical section. A web thickness is selected of the nearest convenient dimension below calculated value. The web area is then deducted from total, and the residue is allocated to top and bottom flanges in the ratio $1 : a$. A flange thickness (or average value) is chosen, at the nearest value below that calculated, and the necessary widths are obtained. Any appreciable cavities can be allowed for by increasing the overall flange areas. It remains to calculate the actual properties of the section, and to check f-values and stresses.

(*b*) *Flanged Section, Top Flange Width Specified*

The method of selection is similar, except that the ratio $\frac{'B}{D} = e$ cannot be assumed, since $'B$ is fixed. In equation (33), B was previously expressed as a function of D; it is necessary here to express it as a function of $'B$.

$$\frac{B}{'B} = \frac{B}{eD} \text{ by definition}$$

$$= \frac{B}{D} \text{ for } e = 1.$$

B is easily obtained in terms of $'B$ by referring to Table 1 for the $\frac{B}{D}$-value corresponding to $e = 1$. The subsequent procedure is identical with that given before, the only variation being that the equation for D_F in (35) reduces to a quadratic.

(*c*) *Rectangular Section*

The procedure is simplified, in that the proportions of the section are fixed, and the only possible variable is f, the intersection value normally being greater than the minimum value; the interpretation of an intersection less than the minimum f-value is that the section is weaker in carrying M_L (or M_S) than M_F, and the depth may be obtained readily from (23), with $\rho = 1$ and f minimum.

Checking Equations

The following gives a rapid method of checking stresses, once the section is known. Since I_c and $(1 - j)\ D$ are calculated there is no need to use

TABLE 1.—COEFFICIENT VALUES

a	c	$b = 0{\cdot}25$							$b = 0{\cdot}20$						
		j	k	B/D for values of e					j	k	B/D for values of e				
				1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{3}$	$\frac{1}{4}$			1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{3}$	$\frac{1}{4}$
1·0	0	0·5	6·87	0·500	0·333	0·250	0·167	0·125	0·5	6·12	0·400	0·267	0·200	0·133	0·10
	0·1	0·5	7·45	0·550	0·366	0·275	0·183	0·138	0·5	6·85	0·460	0·307	0·230	0·153	0·11
	0·2	0·5	8·01	0·600	0·400	0·300	0·200	0·150	0·5	7·55	0·520	0·347	0·260	0·173	0·13
	0·3	0·5	8·56	0·650	0·433	0·325	0·217	0·162	0·5	8·20	0·580	0·387	0·290	0·193	0·14
	0·4	0·5	9·09	0·700	0·466	0·350	0·233	0·175	0·5	8·83	0·640	0·427	0·320	0·213	0·16
0·75	0	0·446	7·00	0·437	0·292	0·219	0·146	0·109	0·443	6·24	0·350	0·233	0·175	0·117	0·08
	0·1	0·452	7·65	0·487	0·325	0·244	0·162	0·122	0·451	7·08	0·410	0·273	0·205	0·137	0·10
	0·2	0·457	8·30	0·537	0·358	0·269	0·179	0·134	0·457	7·84	0·470	0·313	0·235	0·157	0·11
	0·3	0·460	8·92	0·587	0·392	0·294	0·196	0·147	0·462	8·58	0·530	0·353	0·265	0·177	0·13
	0·4	0·463	9·51	0·637	0·425	0·319	0·212	0·159	0·465	9·26	0·590	0·393	0·295	0·197	0·14
0·50	0	0·375	7·69	0·375	0·250	0·187	0·125	0·094	0·367	6·87	0·300	0·200	0·150	0·100	0·07
	0·1	0·390	8·41	0·425	0·283	0·212	0·142	0·106	0·389	7·77	0·360	0·240	0·180	0·120	0·09
	0·2	0·401	9·12	0·475	0·317	0·237	0·158	0·119	0·405	8·63	0·420	0·280	0·210	0·140	0·10
	0·3	0·411	9·81	0·525	0·350	0·262	0·175	0·131	0·417	9·40	0·480	0·320	0·240	0·160	0·12
	0·4	0·419	10·46	0·575	0·383	0·287	0·192	0·144	0·426	10·15	0·540	0·360	0·270	0·180	0·13
0·333	0	0·312	9·02	0·333	0·222	0·167	0·111	0·0833	0·300	8·11	0·267	0·178	0·133	0·089	0·06
	0·1	0·337	9·72	0·383	0·256	0·192	0·128	0·096	0·337	8·91	0·327	0·218	0·163	0·109	0·08
	0·2	0·356	10·39	0·433	0·289	0·217	0·144	0·108	0·362	9·73	0·387	0·258	0·193	0·129	0·09
	0·3	0·371	11·08	0·483	0·322	0·242	0·161	0·121	0·381	10·49	0·447	0·298	0·223	0·149	0·11
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—
0·25	0	0·275	10·50	0·312	0·208	0·156	0·104	0·078	0·260	9·47	0·250	0·167	0·125	0·0833	0·06
	0·1	0·306	10·99	0·362	0·242	0·181	0·121	0·0905	0·307	10·00	0·310	0·207	0·155	0·103	0·07
	0·2	0·330	11·56	0·412	0·276	0·206	0·138	0·103	0·338	10·67	0·370	0·247	0·185	0·123	0·09
	0·3	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—
0·20	0	0·250	12·0	0·300	0·200	0·150	0·100	0·075	0·233	10·92	0·240	0·160	0·120	0·080	0·06
	0·1	0·286	12·20	0·350	0·233	0·175	0·117	0·0875	0·287	11·00	0·300	0·200	0·150	0·100	0·07
	0·2	0·313	12·60	0·400	0·267	0·200	0·133	0·100	0·323	11·56	0·360	0·240	0·180	0·120	0·09
	0·3	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—

$(1 - j)$ and K, and similarly the stresses are used in place of the stress factors θ, ρ, and δ. c_{bL} is zero, as it has been assumed throughout.

By equations (23) and (27):

$$\frac{M_F}{c_a B D^2} = \frac{\theta\delta}{K(1-j)} - \frac{\lambda M_L}{c_a B D^2}$$

hence

$$\theta\delta c_a = \frac{K(1-j)(M_F + \lambda M_L)}{BD^2}$$

or

$$'c_{bs} = \frac{(1-j)D}{I_c}(M_F + \lambda M_L) \quad . \quad . \quad . \quad . \quad . \quad (36)$$

This equation enables $'c_{bs}$ to be checked

By equation (26):

$$\rho = \frac{\left(\frac{'c_{bs}}{c_a} + \frac{\theta\delta j}{(1-j)}\right)}{(1+\lambda)}$$

TABLE 1.—COFEFICIENT VALUES (CONTINUED)

		$b = 0{\cdot}15$							$b = 0{\cdot}10$						
				B/D for values of e							B/D for values of e				
	e	j	k	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{3}$	$\frac{1}{4}$	j	k	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{3}$	$\frac{1}{4}$
	0	0·5	5·49	0·300	0·200	0·150	0·100	0·075	0·5	4·92	0·200	0·133	0·100	0·067	0·050
	0·1	0·5	6·43	0·370	0·247	0·185	0·123	0·0925	0·5	6·23	0·280	0·187	0·140	0·093	0·070
·0	0·2	0·5	7·28	0·440	0·293	0·220	0·147	0·110	0·5	7·33	0·360	0·240	0·180	0·120	0·090
	0·3	0·5	8·06	0·510	0·340	0·265	0·170	0·132	0·5	8·24	0·440	0·293	0·220	0·147	0·110
	0·4	0·5	8·77	0·580	0·387	0·290	0·193	0·145	0·5	9·01	0·520	0·347	0·260	0·173	0·130
	0	0·439	5·60	0·262	0·173	0·131	0·087	0·0655	0·436	5·02	0·175	0·117	0·087	0·0583	0·0437
	0·1	0·452	6·66	0·332	0·223	0·166	0·111	0·083	0·456	6·50	0·255	0·170	0·127	0·085	0·0638
·75	0·2	0·461	7·62	0·402	0·268	0·201	0·134	0·101	0·467	7·68	0·335	0·223	0·167	0·112	0·082
	0·3	0·466	8·45	0·472	0·315	0·236	0·157	0·118	0·473	8·63	0·415	0·277	0·207	0·138	0·104
	0·4	0·471	9·24	0·542	0·362	0·271	0·181	0·136	0·477	9·45	0·495	0·330	0·247	0·165	0·124
	0	0·358	6·12	0·225	0·150	0·112	0·075	0·0562	0·350	5·54	0·150	0·100	0·075	0·050	0·0375
	0·1	0·392	7·29	0·295	0·197	0·147	0·098	0·0737	0·402	7·06	0·230	0·153	0·115	0·077	0·0575
50	0·2	0·413	8·32	0·365	0·243	0·182	0·122	0·0912	0·428	8·32	0·310	0·207	0·155	0·103	0·0775
	0·3	0·427	9·21	0·435	0·290	0·217	0·145	0·109	0·442	9·29	0·390	0·260	0·195	0·130	0·0975
	0·4	0·437	10·00	0·505	0·337	0·252	0·168	0·126	0·452	10·11	0·470	0·313	0·235	0·157	0·1175
	0	0·287	7·29	0·200	0·133	0·100	0·0667	0·050	0·275	6·56	0·133	0·0889	0·0667	0·0444	0·0333
	0·1	0·343	8·29	0·270	0·180	0·135	0·090	0·0675	0·360	7·86	0·213	0·142	0·1067	0·0711	0·0533
333	0·2	0·375	9·22	0·340	0·227	0·170	0·113	0·085	0·398	9·02	0·293	0·196	0·147	0·0978	0·0733
	0·3	0·396	10·10	0·410	0·273	0·205	0·137	0·1025	0·420	9·95	0·373	0·249	0·187	0·1244	0·0933
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	0·244	8·58	0·188	0·125	0·094	0·0625	0·047	0·230	7·68	0·125	0·0827	0·0625	0·0413	0·0312
	0·1	0·314	9·14	0·258	0·172	0·129	0·086	0·0644	0·336	8·50	0·205	0·136	0·1025	0·0681	0·0512
·25	0·2	0·354	9·95	0·328	0·218	0·164	0·109	0·082	0·382	9·54	0·285	0·190	0·142	0·095	0·0712
	0·3	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0	0·217	9·82	0·180	0·120	0·90	0·060	0·045	0·200	8·93	0·120	0·080	0·060	0·040	0·030
	0·1	0·296	9·88	0·250	0·167	0·125	0·0833	0·0625	0·320	9·02	0·200	0·133	0·100	0·0667	0·050
·20	0·2	0·341	10·52	0·320	0·213	0·160	0·107	0·080	0·371	9·90	0·280	0·187	0·140	0·0933	0·070
	0·3	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	0·4	—	—	—	—	—	—	—	—	—	—	—	—	—	—

If $'c_{ts} = c_{at}$ (the allowable tensile stress in the concrete) it may be deduced that :

$$c_{tL} = \frac{1}{(1+\lambda)}\left\{c_{at} + c_{bs}\frac{jD}{(1-j)D}\right\} \quad . \quad . \quad . \quad (37)$$

Thus by (37) the value of c_{tL} may be calculated, corresponding to $'c_{tS} = c_{at}$ (or 0 if required), and this value is the minimum that can be used, provided that the corresponding value of f is satisfactory. It is desirable to keep c_{tL} to a minimum, since for a given section the steel force is directly proportional to c_{tL}.

Equation (17) may be written :

$$M_L = \frac{c_{tL}}{D}\{I_c + A_c(1-j)D(1-j-f)D\} \quad . \quad . \quad (38)$$

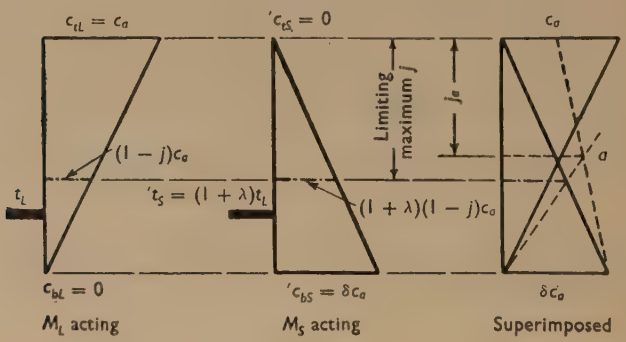
From this equation, it is evident that for a given section and moment, c_{tL} is a minimum when $(1-j-f)D$ is maximum, and hence using the minimum value of c_{tL} calculated from equation (37), the corresponding

value of $(1 - j - f)D$ is calculated from (38). If this value is satisfactory it should be adopted, whereas if it is too high, then using the maximum possible value, the corresponding c_{tL} value is obtained from (38). Obviously c_{tL} must not exceed c_a.

Having checked the section for concrete stresses, the steel force under M_L can be calculated (being $A_c p t_L$) from equation (16). The steel area may be obtained by assuming a suitable value of t_L, or preferably by specifying a factor of safety, and calculating it.

In the method of selecting a section, reference was made to a limiting maximum value of j; the reason for this is that, as was pointed out, since a relation $'t_S = t\ (1 + \lambda)$, and the three stresses c_{tL}, c_{bL}, and $'c_{bS}$ are assumed, it is not possible to assume $'c_{tS}$ for any section. Referring to

Figs 7



$$\frac{(1 + \lambda)(1 - j)c_a}{jD} = \frac{\delta c_a}{D} \quad \therefore j = \frac{1 + \lambda}{1 + \lambda + \delta}$$

For any lesser value such as j_a, with c_{tL}, c_{bL}, and $'c_{bS}$ specified, the stress line under M_S passes through a and necessitates $'c_{tS}$ to be greater than zero, and vice versa.

Fig. 7, it will be observed that if $'c_{tS} = 0$ is assumed as a limiting value, the value of j is fixed, its value being obtained from:

$$\frac{\delta c_a}{D} = \frac{(1 - j)(1 + \lambda)c_a}{jD}$$

hence

$$j = \frac{(1 + \lambda)}{(1 + \lambda + \delta)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

Inspection shows that any higher value of j makes $'c_{tS}$ tension, and hence the value given by equation (39) may be taken as an upper limiting value.

Beams Subject to Inversion

Where, in the course of handling, it is possible for a prestressed beam to be inverted from its normal position, it is necessary to design for this, the treatment being similar. It will be realized that unless the self-weight is small this is somewhat uneconomical.

The two extreme moments applied to the section are now M_L and $M_I = -M_G$ = the inverted self-weight moment. Equation (27) expresses the change of moment between the two extreme conditions of loading, in terms of the stresses; for this case, however, the change of moment is $M_L - M_I = M_L + M_G$, instead of $M_L - M_S = M_F$. By analogy, it can be seen that:

$$\frac{(M_L + M_G)}{c_a BD^2} = \frac{\theta\delta}{K(1-j)} - \rho\lambda\left\{\frac{1}{K} + (1-j)(1-j-f)\right\}. \quad (40)$$

With M_F as defined, that is, $M_F = M_L - M_S$ and $M_S = M_G$, that is, $M_A = 0$

$$M_F = 2M_L - (M_L + M_G).$$

Eliminating M_L and M_G, using (23) and (40), the following is divided:

$$\frac{M_F}{c_a BD^2} = \left\{\frac{1}{K} + (1-j)(1-j-f)\right\}(2+\lambda) - \frac{\theta\delta}{K(1-j)}. \quad (41)$$

In this it is evident that λ must be defined by $'t_I = (1+\lambda)t_L$ where $'t_I$ denotes the steel stress under M_I, under instantaneous conditions.

Substituting $\frac{M_G}{c_a BD^2} = \frac{1{\cdot}56L^2}{c_a D}$ in (40) and replacing M_L by (23), the following is obtained:

$$\frac{1{\cdot}56L^2}{c_a D} = \frac{\theta\delta}{K(1-j)} - \rho(1+\lambda)\left\{\frac{1}{K} + (1-j)(1-j-f)\right\}. \quad (42)$$

Equations (41) and (42) are used in a similar manner to (33) and (34), taking ρ and θ as unity for the purpose of selecting a section; if the values of D so obtained are D_F and D_I respectively, then it will be observed that increase of f increases D_F and decreases D_I.

Since the moment on the section is reversible, it is evident that the f-value will rarely be the minimum that could be used.

Checking Equations

By (41) $$c_{bI} = \frac{(1-j)D}{I}\{M_L(2+\lambda) - M_F\} \quad . \quad . \quad . \quad (43)$$

By analogy with (37):

$$c_{tL} = \frac{1}{(1+\lambda)}\left\{c_{at} - {}'c_{bI}\frac{jD}{(1-j)D}\right\} \quad . \quad . \quad . \quad (44)$$

This gives the minimum c_{tL} value.

The Paper is accompanied by seven sheets of diagrams, from which the Figures in the text have been prepared.

Paper 5890

"The Theory and Design of Axially Loaded Prestressed Concrete Tension Members"

by

Frederick William Gifford, B.Sc.(Eng.), Ph.D.

Ordered by the Council to be published with written discussion.†

SYNOPSIS

The theory of axially loaded prestressed concrete tension members is developed, taking into account the strains which occur, and allowing for creep and shrinkage. The "exact" solution is shown to be unsuitable for deriving the concrete section, but can be used to check a section if obtained by another means.

Two simple methods are given for calculating the concrete section required, and thus obviate the need for using trial-and-error.

Two occasions will arise when it is desirable to use axially loaded prestressed tension members—first, in order to use very-high-tensile steels for the purpose of steel economy, when an ordinarily reinforced member would crack excessively; and secondly, where cracking is undesirable, as in liquid-retaining structures.

Two extreme conditions of loading must be considered, namely, maximum and minimum external tension; the former produces minimum compressive concrete stresses, the latter maximum. In addition, there are two limiting conditions of creep and shrinkage—first, the "instantaneous" condition at the time of transfer of stress on to the concrete, when the effects are minimum; and also "full creep and shrinkage," when after a period of time the effects are maximum. Creep and shrinkage result in a gradual reduction of steel stress, and consequently of concrete stress.

Clearly, then, maximum concrete stress corresponds to minimum axial tension coupled with the "instantaneous" condition of creep and shrinkage, whilst minimum concrete (compression) stress corresponds to maximum axial tension, with full creep and shrinkage.

Tests on prestressing steels show that if a steel is maintained under a constant strain, the stress falls off, but with time it reaches a limiting value; this drop in stress corresponds to a residual creep strain.

† Correspondence on this Paper should be received at the Institution by the 15th April, 1954, and will be published in Part III of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

Similarly, if concrete is maintained under a constant stress, in addition to the normal instantaneous elastic strain, creep strain takes place with time, reaching a limiting value; also, for the working range of concrete stresses, it has been found that the limiting creep strain is approximately proportional to the stress. It is thus possible to define two values of E for concrete:

${}_{i}E_c$ to denote the live-load E of concrete (purely elastic strains).
${}_{d}E_c$,, the dead-load E of concrete (elastic and total creep strains).

Thus in the following analysis, when dealing with instantaneous stresses, ${}_{i}E_c$ is used, and when dealing with the full-creep-and-shrinkage condition, ${}_{d}E_c$ is used for all sustained loadings—that is, prestress and dead loads.

Shrinkage of concrete is independent of stress, and reaches a limiting maximum value with time, which may be expressed as a strain.

List of Symbols

(See *Fig. 1*)

c denotes concrete stress, compression positive.
t ,, steel stress, tension positive.
N ,, axial tensile force applied to member.

The following are used as suffixes to N, c, and t to indicate the particular tension acting, and the corresponding stresses in the concrete and the steel of the prestressed member:

S to denote minimum sustained (dead) load acting at the time of transfer of stress to the concrete.
D ,, the total dead load to be applied.
L ,, the total working (dead plus live) load.
U ,, the ultimate load.

To differentiate between the conditions of instantaneous stresses and full creep and shrinkage, a prefix $'$ is used to indicate the former condition.

E_s denotes the modulus of elasticity of the steel.
m_l and m_d denote respectively the live-load and dead-load modular ratios corresponding to ${}_{i}E_c$ and ${}_{d}E_c$.
Δ_{cs} denotes the total creep strain of the steel.
Δ_{sh} ,, the total shrinkage strain of the concrete.
Δ_{cs}' ,, the steel creep strain occurring before transfer of stress to the concrete.
Δ_{sh}' ,, the concrete shrinkage strain occurring before transfer of stress to the concrete.

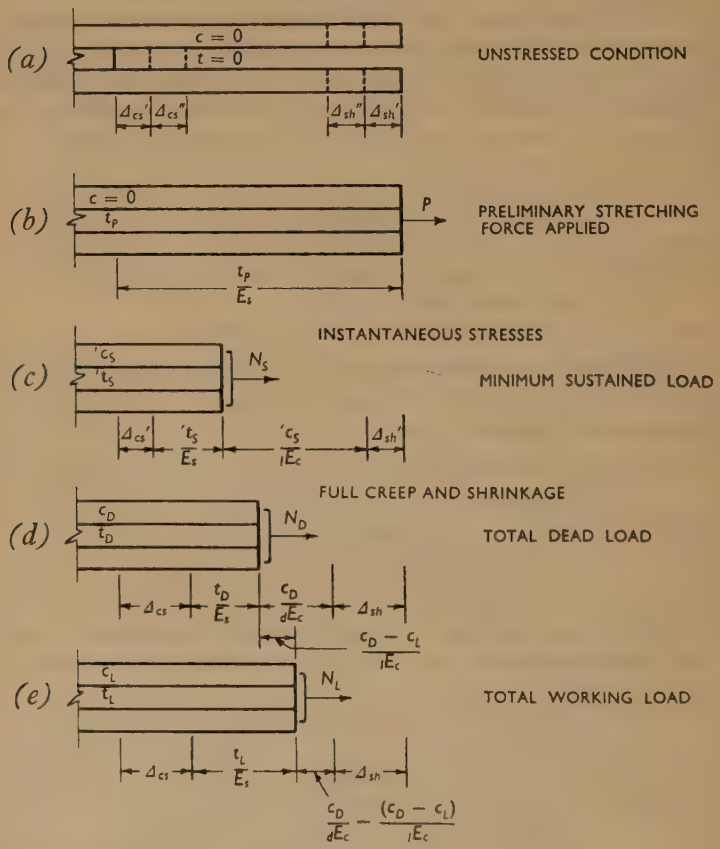
$$\Delta_{cs}'' = \Delta_{cs} - \Delta_{cs}' \qquad \Delta_{sh}'' = \Delta_{sh} - \Delta_{sh}'$$

c_a and δc_a denote allowable concrete compressive stresses at 28 days and at time of transfer to concrete, respectively.

Bonded Construction

The diagrams in *Figs 1* are intended to represent the relative movements of one end of the concrete and steel, the far end being fixed. The movements are expressed as strains, the length being sensibly constant. *Fig. 1* (*a*) shows the relative lengths of the members, in an unstressed condition,

Figs 1



the dotted lines indicating the ends of the members for the relevant strains induced by shrinkage of concrete and creep of steel; it will be realized that the latter cannot occur while unstressed, but theoretically it is possible to stress the members and subsequently release the stress, the residual strains being those shown. *Fig. 1* (*b*) shows the steel stretched and held by some external reaction, with the concrete unstressed. *Figs 1* (*c*), *1* (*d*),

and *1* (*e*) are the relevant equilibrium strain diagrams under external loads.

The following strain equations are readily derived from *Figs 1*:

$$t_P = {}'t_S + m_l \,.\, {}'c_S + E_s(\Delta_{cs}' + \Delta_{sh}') \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$t_P = t_D + m_d \,.\, c_D + E_s(\Delta_{cs} + \Delta_{sh}) \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$t_P = t_L + m_{cr} \,.\, c_D + m_l \,.\, c_L + E_s(\Delta_{cs} + \Delta_{sh}) \quad . \quad . \quad (3)$$

where $m_{cr} = m_d - m_l$.

Considering forces, the following may be obtained:

$$P = A_s \,.\, t_P \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$A_s \,.\, {}'t_S = N_S + A_c \,.\, {}'c_S \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$A_s \,.\, t_D = N_D + A_c \,.\, c_D \quad . \quad . \quad . \quad . \quad . \quad (6)$$

$$A_s \,.\, t_L = N_L + A_c \,.\, c_L \quad . \quad . \quad . \quad . \quad . \quad (7)$$

$$A_s \,.\, t_U = N_U \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

the latter being from ultimate load conditions, the concrete being considered cracked and carrying no load.

In design, the following terms can be specified or assessed:

t_U N_U N_L N_D N_S m_l m_d m_{cr} Δ_{cs} Δ_{cs}' Δ_{sh} Δ_{sh}' E_s ${}'c_S$ ($= \delta c_a$, say) c_L ($= 0$, say).

This leaves the following unknowns:

P t_P t_L ${}'t_S$ t_D A_c A_s c_D

Thus there are eight equations in eight unknowns, which theoretically may be solved direct; evidently if one of these is assumed, then, since loads and properties of materials are fixed, c_L and ${}'c_S$ cannot both be assumed.

The problem in design is to find A_c and A_s, the latter being readily obtained from equation (8). A_c may be obtained by:

(i) guessing;
(ii) using an approximate method; or
(iii) obtaining a single equation in A_c

Any A_c may be checked by the following method:

from equation (8) obtain A_s
,, ,, (7) specify c_L and obtain t_L
,, ,, (2) and (3) $t_L - t_D = (c_D - c_L)m_l$ hence obtain ratio $c_D : t_D$
,, ,, (6) obtain c_D and t_D
,, ,, (2) obtain t_P
,, ,, (1) obtain ${}'t_S : {}'c_S$
,, ,, (5) obtain ${}'t_S$ and ${}'c_S$. The latter must be within the allowable range for the A_c to be satisfactory. Finally,
,, ,, (4) calculate P.

Inaccurate guessing will result in unnecessary calculation, and on the other hand the exact solution is cumbersome ; the following approximate methods can be recommended :—

(*a*) Assumption of ratio of steel stresses, as in bending ; that is, in $'t_S = (1 + \lambda)t_L$, the factor λ is assumed from past experience. Using this relation in conjunction with equations (5) and (7), the following can be derived :

$$A_c = \frac{(1 + \lambda)N_L - N_S}{'c_S - (1 + \lambda)c_L} \quad . \quad . \quad . \quad . \quad . \quad (9)$$

This is particularly satisfactory when previous designs have been made using the same stresses and so on, and λ-values are known.

(*b*) The following equation is a close approximation to the exact solution and can be used without the need for assessing λ :

$$A_c = \frac{(N_L - N_S) + A_s \,.\, E_s(\Delta_{cs}'' + \Delta_{sh}'')}{('c_S - c_L)} - A_s\left[m_d - m_{cr}\left\{\frac{'c_S}{('c_S - c_L)} + \frac{(N_L - N_D)}{(N_L - N_S)}\right\}\right] \quad . \quad (10)$$

End-Anchored Construction

Provided that $p = \dfrac{A_s}{A_c}$ is constant over the full length of the member, then irrespective of whether the member is grouted subsequently or not, all the above equations apply ; it will be realized that the states of stress shown in *Figs 1* (*a*) and *1* (*b*) are imaginary, since in fact the steel is pulled up against the concrete, but nevertheless theoretically the conditions may be considered.

In this case the stress to be initially induced into the steel by the jack, prior to release on to the anchorage, will be $'t_S$ plus an allowance for the pull-in of the anchorage.

In the unlikely case of a variable p, the force equations remain the same, but it will be necessary to introduce into the strain equations a " strain factor " in order to equate the strain in the steel to the average strain in the concrete.

The Paper is accompanied by one sheet of diagrams from which *Figs 1* have been prepared.

Paper No. 5935

"Detailed Analysis of a Discharge Measurement on the Victoria Nile"

by

Clement Leonard Berg, M.I.C.E., A.M.I.Mech.E.

(*Ordered by the Council to be published in abstract form.*) †

Explanatory.

The discharge of the Nile at its source, where it flows out of Lake Victoria in Uganda, is necessarily measured at Namasagali, about 45 miles downstream, because the river above this point flows in a ravine, and is broken by rocks and rapids. In this distance the losses must be small and there are no tributary streams of any importance. Namasagali itself, is not an ideal discharge-measuring site, because at times Lake Kyoga lower down backs up and influences the depth of water. There is, therefore, no reliable relationship between the gauge readings at Namasagali and the discharge, but there is a relationship between the gauge reading on Lake Victoria since the outflow is over the Ripon Falls, which form a natural weir.

The "half-depth" method has been adopted in recording discharge measurements, not only at Namasagali but throughout the Nile Basin. This method consists of lowering a current meter to half the depth of the water at a number of equidistant points along a wire rope stretched tightly across the river at right angles to the flow, multiplying the velocity thus obtained by the cross-sectional area of the corresponding section, adding the results together, and finally multiplying the total by a constant 0·96. It is, of course, necessary to multiply the total by a constant, because the average velocity of flow is not obtained at half the depth of a river, and it has been found on inspection of a large number of velocity-depth curves that the factor of 0·96 used in the half-depth method is, on the average, accurate. This agrees with a large number of measurements made on many rivers in the United States and elsewhere.

Various authorities have stated, however, that in actual recordings the depth of mean velocity may vary from 0·5 to 0·7 of the total depth; a range which may easily correspond to as great a difference as 20 per cent in the velocity. Since such diverse results are met in practice, it is not advisable to apply a result obtained by an average from a large number

† The full MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

of cases to a particular case if accuracy is the aim. In the case of Namasagali accuracy is of great importance, for it is on the results obtained from this site that the discharge from Lake Victoria is calculated, and these results will influence the control of the discharge once the dam is completed for the Owen Falls hydro-electric scheme.

The first discharge measurement was taken at Namasagali in March 1923, and since then measurements have been taken at various times. They are now recorded regularly every week. From these results a formula has been devised for the discharge in relation to the Lake Victoria gauge readings, and since the latter exist for the past 56 years, Tables have been worked out showing the 10-day mean discharge covering the whole of this period, thus giving them the appearance of authenticity which they may not deserve.

Observations

The width of the water spread across the river at Namasagali, when the present measurements were recorded, was 465 metres, measured along the wire rope. The first observation was taken 30 metres from the right bank in order to clear the weeds growing along the edge; thereafter observations were recorded every 25 metres, except the last which had to be 10 metres from the previous one because a thick expanse of Papyrus growth was encountered on the opposite bank through which it has been assumed no water percolates. Thus, observations were recorded at nineteen points across the river, and at each point many velocity measurements were taken at various depths—from the water surface down to the river-bed. In all, 193 velocity measurements were recorded during the observations. A velocity/depth curve was drawn to a large scale for each point of observation, from which the average velocity and the half-depth velocity were determined. The results are shown in Table 1.

The observations were carried out with a new Ott current-meter having a rating Table certified by the Hydrologic Laboratory of the Mathematical and Mechanical Institute of Bavaria. A double check was made, however, with two other current-meters, namely a new Amsler having a rating Table certified by the Swiss Federal Department of Water Development, and an old Gurley having a rating Table certified by the Physical Department of the Government of Egypt. The results obtained from these three different current-meters all agreed remarkably well; in fact, the overall difference between the two extremes was within 0·6 per cent which, it is hoped, will encourage confidence in the conclusions to be drawn from this experiment.

Fortunately the observations were carried out when the discharge was very near to the average discharge of Lake Victoria over a large number of years. The measured discharge by the half-depth method was 619 cumecs, and the average discharge by the same method is 632 cumecs. It can therefore be assumed with confidence, that although the result may not

TABLE 1.—DISCHARGE FORM FOR OTT CURRENT-METER NO. 7799

Computation of Discharge—19th November, 1952

Chainage : metres	Distance between soundings : metres	X ½ sum of neighbouring distances : metres	Y Soundings : metres	Area ($A = XY$) : sq. m.	Average velocity, V : m./sec.	$A \times V$: cubic metres	Velocity at half depth, V_1 : m./sec.	$A \times V_1$: cubic metres
0			0	0	0	0	0	0
30	30	27·5	1·33	36	0·050	2	0·030	1
55	25	25	1·76	44	0·276	12	0·327	14
80	25	25	2·16	54	0·395	21	0·428	23
105	25	25	2·51	63	0·393	25	0·440	28
130	25	25	2·93	73	0·421	31	0·470	34
155	25	25	3·29	82	0·381	31	0·416	34
180	25	25	3·74	94	0·484	45	0·545	51
205	25	25	4·03	101	0·522	53	0·561	57
230	25	25	4·34	108	0·527	57	0·572	62
255	25	25	4·43	111	0·538	60	0·576	64
280	25	25	4·04	101	0·507	51	0·564	57
305	25	25	3·56	89	0·526	47	0·559	50
330	25	25	3·10	78	0·501	39	0·548	43
355	25	25	2·39	60	0·437	26	0·506	30
380	25	25	1·87	47	0·396	19	0·462	22
405	25	25	1·69	42	0·379	16	0·414	17
430	25	25	2·14	54	0·369	20	0·396	21
455	25	17·5	1·63	28	0·271	8	0·308	9
465	10	5	1·57	8	0·172	1	0·190	2
					Discharge	564	Total	619

Half-depth discharge = 619 × 0·96 = 594

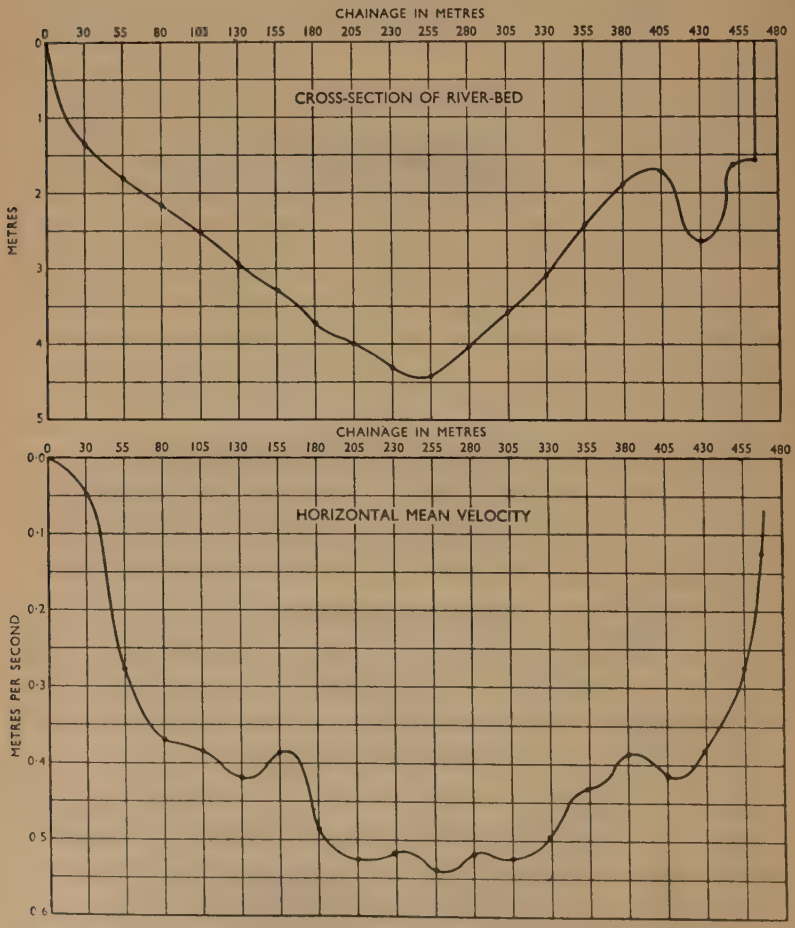
strictly apply when the discharge is either very much more or very much less, yet on the average it can be considered reliable.

Results

The cross-section of the river bed and the horizontal mean velocity curve are shown in *Figs 1.* This curve is typical and would indicate that the observations have been taken without appreciable error.

Table 1 is self explanatory. The discharge worked out by using the average velocity will, of course, give the correct result, which is 564 cumecs. The corresponding figure obtained by using the half-depth method with

Figs 1



Cross-Section of River Bed and Horizontal Mean Velocity Curve

the constant 0·96 is $619 \times 0{\cdot}96 = 594$ cumecs, that is 5·3 per cent in excess. In order to obtain the correct result the constant should be 0·91 for the Namasagali discharge site and not 0·96.

It was also found, by careful survey, that the width of the water spread across the river was only 462·3 metres and not 465 metres as measured along the inevitably sagging wire rope. This would account for a further error in the result of approximately 0·6 per cent in excess. All the discharge measurements recorded at Namasagali would therefore appear to have been in the neighbourhood of 6 per cent in excess of the correct figures, and the average discharge of Lake Victoria over many years would be 596 cumecs and not 632 cumecs as previously stated.

The Paper is accompanied by seven sheets of diagrams, from one of which *Figs 1* have been prepared, and by twenty-one Tables.

CORRESPONDENCE

on a Paper published in Proceedings, Part III, April 1953

Paper No. 5875

" The Generalized Gould's Function " †

by

Robert Mathieson, B.Sc., Ph.D.

Correspondence

Professor Steponas Kolupaila, of Notre Dame University, Indiana, observed that the Author had done valuable work in calculating the Tables of integrals for Gould's method of flood routing through a reservoir. Those values were given for a few typical cross-sections of the outlet; they could be extrapolated for intermediate exponents, corresponding to any actual form of cross-section.

Professor Kolupaila pointed out that some of the calculated values were exactly the same as those widely used for computation of the varied flow in open channels. B. Bakhmeteff [1] had published similar Tables for exponents 2·8 to 4·2; N. N. Pavlovskij [2] had computed generalized Tables for exponents 2·0 to 5·5. Both had explained different ways of integration of infinite series. Those Tables were also published in Professor Kolupaila's "*Hidraulika.*" [3] He thought that the methods used by those authors mentioned would be of interest to Dr Mathieson.

The Author, in reply, agreed with Professor Kolupaila that the first of the three functions tabulated, $\phi(r)$, was identical with the well-known variable-flow function $B(\eta)$ for which Professor Bakhmeteff had published values for exponents 2·8 to 5·4 in 1932. Since, however, only one of the exponents (3·5) for which $\phi(r)$ was tabulated fell within that range and even then would have involved interpolation, it was considered more appropriate to calculate the particular values concerned. The Author was interested to learn of the additional works mentioned.

† Proc. Instn Civ. Engrs, Part III, vol. 2, p. 142 (Apr. 1953).

[1] B. A. Bakhmeteff, "Hydraulics of open channels." McGraw-Hill, 1932. *See* pp. 303–313.

[2] N. N. Pavlovskij, "*Kratkij gidravlicheskij spravochnik.*" Leningrad, 1940.

[3] S. Kolupaila, "*Hidraulika.*" Kempten, 1947. *See* pp. 246–259.

CORRESPONDENCE

on Papers published in Proceedings, Part III, August 1953

Structural Paper No. 34

" Corrugated Concrete Shell Roofs " †

by

James Hardress de Warrenne Waller, D.S.O., O.B.E., M.E., M.Sc., M.I.C.E.

and

Alan Clift Aston, B.Sc.(Eng.)

Correspondence

Captain J. H. Foster observed that for the past 4 years he had been advocating the employment of gun application, with reference to high-speed construction by skilled craftsmen. He had himself used the method on a 30-foot-span cellular concrete shell, 1 inch thick, and a 40-foot-span cement shell, 1¼ inch thick, with results markedly superior to those obtained by hand application. He also advocated a second attempt at a gun-applied gypsum-plaster 30-foot-span shell.

Having established the feasibility of gunning, Captain Foster considered that the most useful application of light shells would be as low-crowned arches in conjunction with prestressed concrete valley beams. Details of an actual scheme, with estimates and comments made by the Building Research Station, were given in a Paper * entitled " Shell Concrete "' which he had presented to the Institution in September 1952.

He believed firmly that the Ctesiphon principle could be adapted so as to be practically indistinguishable to the layman from normal barrel vaulting. With a multi-span lay-out of 30 feet by 60 feet or 25 feet by 50 feet, he considered that whilst the costs would be comparable with that of steel construction (substantially lower than barrel vaults), there was no real competition since the lay-outs were generally believed to be uneconomic for barrel vaults.

Captain Foster regretted that the Ctesiphon principle expounded by the Authors had not enjoyed a greater popularity in the United Kingdom,

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† Proc. Instn Civ. Engrs, Part III, vol. 2, p. 153 (Aug. 1953).

* The MS. and illustrations may be seen in the Institution Library.—Sec. I.C.E.

and he thought that the Authors would be well advised to put more emphasis on the advantages of multi-span construction of the range mentioned above.

The Authors, in reply, stated that Captain Foster's proposal to use spans of 25 or 30 feet would apply to cases where a low total cube was desired. In other cases such small spans were thought to involve unnecessary complications. The use of prestressed valley beams was thoroughly practicable but the possibility and economy of their being spaced at quite large centres should be kept in mind.

Public Health Paper No. 6

" Controlling Factors in the Choice of Sewage-Treatment Processes for Sewage Disposal " †

by

William Fillingham Brown, B.Sc.(Eng.), M.I.C.E.

Correspondence

Mr G. W. Wilkinson observed, in connexion with the Author's remarks on p. 242 concerning dilution ratios, that a further factor which had to be taken into consideration was the regime of the receiving water. Conditions could arise where the suspended solids content of the effluent, adopted on a dilution ratio basis only, might prove to be unsatisfactory because there were impounded reaches downstream, such as mill-dams, weirs, and ornamental-lakes, where the solids could settle and putrify. He considered that where such conditions occurred they should be given very serious consideration in the design of the final stages of a purification plant.

On the question of humus tanks to meet higher effluent standards, Mr Wilkinson fully agreed with the Author that continuous removal of humus by means of tanks of improved design would be essential. It was all too evident that satisfactory effluents from biological filters were often spoiled during their passage through flat-bottomed humus tanks, because the intermittent hand cleaning of the latter was inadequate.

The continuous withdrawal of return and surplus activated sludge from the final separating tanks of the activated-sludge process resulted in that process, in general, consistently producing effluents of lower solids content than passed from the humus tanks of biological filter plants, from which the sludge was only intermittently removed. In that connexion it

† Proc. Instn Civ. Engrs, Part III, vol. 2, p. 236 (Aug. 1953).

was interesting to note that the effluent from the final separating tanks of the Mogden plant averaged 7 parts per million of suspended solids, which figure might be compared with 16 parts per million in the effluent from flat-bottomed humus tanks following biological filters of another plant, and 5 parts per million after that effluent had subsequently been passed through rapid sand filters and micro-strainers.

Mr Wilkinson further observed that, outside the United States, there were approximately 156 diffused-air activated-sludge plants treating a total dry-weather flow of 310 million gallons per day; 121 of those plants treated an average of 0·45 million gallons per day each, which corresponded to an average population of 15,000 for each plant (based on 30 gallons per head per day).

The remaining 35 plants treated a total of 256 million gallons per day; 14 of them were for partial purification for a total flow of 177 million gallons per day. In addition, there was a large number of mechanical plants, the majority of which dealt with populations of less than 50,000.

In the United States the total number of plants was about 330, of which 140 were diffused-air and 190 mechanical. All the mechanical plants and 56 of the diffused-air plants were for populations of less than 50,000.

It would appear, therefore, that the Author's opinion, which set the lower limit of population for activated-sludge plants at 50,000, was by no means generally shared.

Thus, the fact that the majority of activated-sludge plants were for populations of less than 50,000, and that many plants had been running satisfactorily for periods of up to 25 years, was proof that they had not failed " through lack of skilled labour and adequate resources for maintenance " (p. 252).

Provided an activated-sludge plant was properly designed with sufficient tank capacity and aeration potential, it did not demand " constant attention . . . also the application of a wide variety of skill, knowledge, and experience " (p. 247), to any greater extent than, say, a biological filter plant for the same conditions. Indeed, the operation of the latter type of plant, particularly where recirculation or alternating double filtration were practised, might demand a great deal of attention in order to secure the desired results.

Mr Wilkinson agreed with the Author that mechanical dewatering and heat drying or incineration of sludge should be reserved for the larger undertakings, but felt that such processes had yet to be proved efficient and economical, even for large plants in Great Britain.

In that connexion, the results obtained from the plant at present operating under the Author's direction would be awaited with interest.

The Author, in reply, stated that he agreed with Mr Wilkinson that, under the conditions cited, special consideration should be given to the suspended-solids content of the effluent, and consequently to the design of the final stages of purification. In many such cases, the Royal Commission

standard would not be high enough to prevent trouble from settlement of humus and subsequent putrefaction and pollution of the stream, though it should be borne in mind that the solid matter associated with well-oxidized effluents was itself reasonably stable, and would give rise to much less pollution than the solid matter associated with insufficiently oxidized effluents.

The Author was aware that there were many activated sludge plants in existence serving populations less than 50,000, but he was still of the opinion that, in general, percolating filters were more economical for plants serving populations below that figure. He did not agree with Mr Wilkinson that no more attention or skill was required in the operation and maintenance of activated-sludge plants than was required for filter installations, especially if the mechanical equipment associated with each type was included. In the Author's opinion, the greater degree of mechanization involved in the activated-sludge system led inevitably to more frequent attention by more highly skilled operatives than would be needed for filter installations, and furthermore, the activated sludge process itself required a greater degree of skilled control by trained chemists having laboratory equipment at their disposal for the purpose. For small individual works, such skilled control could never be provided economically, though it could be made so under regional control of a number of such works.

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